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ISSUED QUARTERLY

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OF THE
New England Water Works Association
1955 - 1956

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THE Association was organized in Boston, Mass., on June 21, 1882, with the object of providing its members with means of social intercourse and for the exchange of knowledge pertaining to the construction and management of water works. From an original membership of only TWENTY-SEVEN, its growth has prospered until now it includes the names of over 900 men. Its membership is divided into six classes, viz.:

A Member shall be an officer or employee of a public or private water works, an engineer, chemist or other person qualified to aid or interested in the advancement of knowledge relative to water works.

An Honorary Member shall be a person of acknowledged eminence in some branch of water supply or of engineering.

A Junior shall be not less than eighteen years nor more than twenty-five years of age, a student or connected with water supply work.

An Associate shall be either a person, firm or corporation engaged in manufacturing or furnishing materials or supplies for the construction or maintenance of water works.

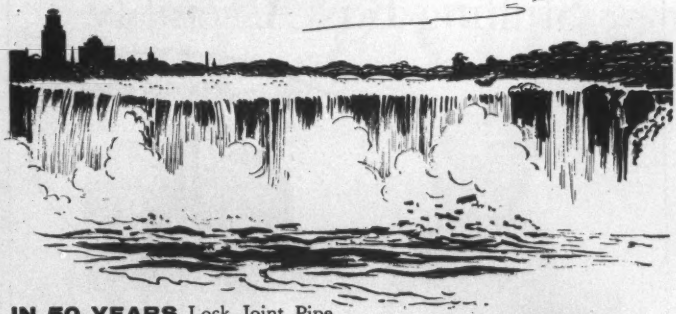
A Corporate Member shall be either a Water Board, Commission, Company or Municipal Corporation.

The initiation fees and annual dues are as follows:

	Initiation Fees		*Annual Dues*
Members	$ 3.00	Members	$ 5.00
Juniors	1.00	Juniors	3.00
Associates	10.00	Associates	20.00
Corporate Members	10.00	Corporate Members	10.00

This Association has at least eight regular meetings each year, of which five are held in Boston, one in northern New England, one in southern New England, and one, the annual convention, held in September or October on such date as the Executive Committee may designate.

580 times more water than flows over Niagara Falls!



IN 50 YEARS Lock Joint Pipe Company has developed and produced concrete pressure pipe for countless water works installations throughout the world. Today, at any given second of the day or night, these pipes are transporting better than a billion gallons of water — or more than 580 times as much water as is flowing over Niagara Falls.

This is the measure of the tremendous contribution Lock Joint Pipe has made, during a half century, toward meeting one of civilization's greatest problems — water supply.

You may be sure that every resource of skill, experience and research at our command is now at work on the answers to the even greater problems we must meet in the *next* 50 years!

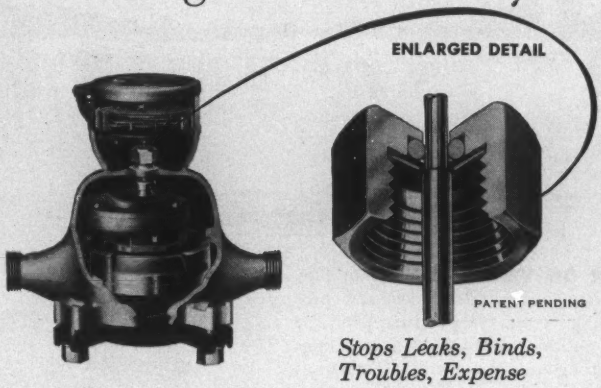
LOCK JOINT PIPE COMPANY

Established 1905

P. O. Box 269, East Orange, N. J.

Sales Offices: Chicago, Ill. • Columbia, S. C. • Denver, Col.
Detroit, Mich. • Hartford, Conn. • Kansas City, Mo.

The Rockwell "O" -Ring Stuffing Box Assembly



Stops Leaks, Binds, Troubles, Expense

The Greatest Single Advance In Water Meter Construction In Years

Here's another outstanding development from Rockwell Research. It's a sure cure for all the headaches that go with meter stuffing box maintenance. Perfected four years ago, this Rockwell "O"-Ring Stuffing Box assembly has now been thoroughly field tested. *It won't leak; can't bind!* Meters with this construction run more freely, stay accurate longer. Worn spindles are a thing of the past.

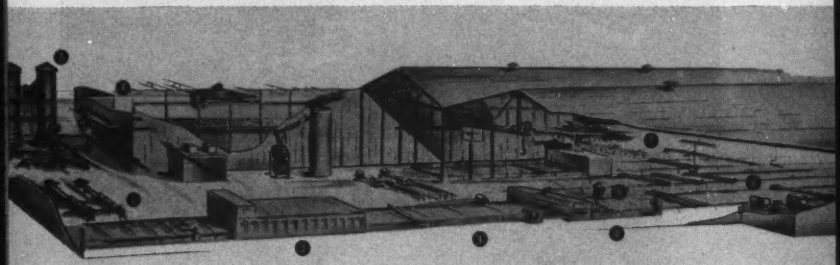
Now *all* Rockwell water meters make use of this *exclusive* "O"-Ring design upon which patent application has been made. There's no extra cost for this construction, but a lot of extra satisfaction and value. Ask your Rockwell representative to demonstrate. And be sure to inquire about the use of "O"-Ring stuffing box nuts as interchangeable replacements for stuffing box assemblies in earlier model Rockwell meters.

ROCKWELL MANUFACTURING COMPANY

PITTSBURGH 8, PA. Atlanta Boston Charlotte Chicago Dallas Denver Houston Los Angeles Midland, Texas. New Orleans New York N. Kansas City Philadelphia Pittsburgh San Francisco Seattle Shreveport Tulsa. In Canada: Rockwell Manufacturing Company of Canada, Ltd., Toronto, Ontario

We are ready for another CENTURY

with our new, ultra-modern WARREN-SPUN pipe shop now being erected in Phillipsburg, N.J.



1. CUPOLA
2. PIPE SPINNERS
3. ANNEALING FURNACE
4. GRINDERS
5. TAR DIP TANK
6. HYDRAULIC TESTERS
7. CEMENT LINING MACHINE
8. MECHANICAL JOINT DRILLS
9. MATERIAL STORAGE

100 years ago—WARREN was founded in Phillipsburg, New Jersey, dedicated to manufacturing a *quality cast iron pipe* for transfer of water to a growing nation. For the next 75 years pit-cast pipe in all sizes and in 12-foot laying length served as the industry standard. In fact, even in this atomic age, pit-cast pipe has no peer in larger sizes up to 84" in diameter.

Today—to meet the even greater demand of our rapidly expanding nation, WARREN manufactures in Everett, Massachusetts, *modernized* cast iron pipe, centrifugally cast in sand-lined molds. Also, currently under construction is a deLavaud pipe shop to produce cast iron pipe centrifugally-spun in metal molds in 18- and 20-foot lengths.

That 73 utilities are still using cast iron pipe installed over 100 years ago is proof of high quality and dependability of cast iron pipe.

MODERNIZED cast iron pipe, centrifugally cast, is stronger, tougher, more uniform in quality and therefore more efficient, making it the world's most dependable pipe.

Only Warren, in the United States, casts pressure pipe by the *pit-cast, sand-spun,* and *deLavaud* processes—thus assuring you the right pipe for your particular need.

WARREN

FOUNDRY & PIPE CORPORATION

55 Liberty Street, New York 5, N. Y.

☆ ☆ ☆

Producers of

PIT-CAST PIPE • SAND-SPUN PIPE
DELAVAUD PIPE • PRESSURE PIPE
FITTINGS • SPECIAL CASTINGS
MAGNETITE IRON ORE • CRUSHED STONE & SAND

Founded 1806

In Our Second Century of FLOWING PROGRESS

THE A. P. SMITH MANUFACTURING CO.

produces, FIRE HYDRANTS A. W. W. A. Specification and High Pressure — Compression Type — Dry and Wet Barrel Types. GATE VALVES sizes 2" thru 66", Manual, Hydraulic Cylinder, Electric Motor Operated for Low, Medium and High Pressure Service. INSERTING VALVES sizes 4" thru 48" for inserting under pressure. TAPPING SLEEVE AND VALVES sizes 4" x 2" thru 60" x 48" for branch connections under pressure.

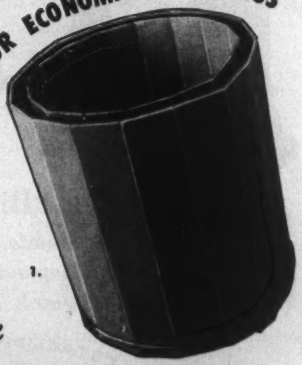
42

THE A.P. SMITH MFG. CO.

EAST ORANGE, NEW JERSEY

1. Large illustration shows Singl-use Fiberform ready to be filled.
2. Filling plate and collar simplify pouring concrete into form.
3. Completed meter box barrel.
4. Meter box with cover in place.

FOR ECONOMICAL SETTINGS



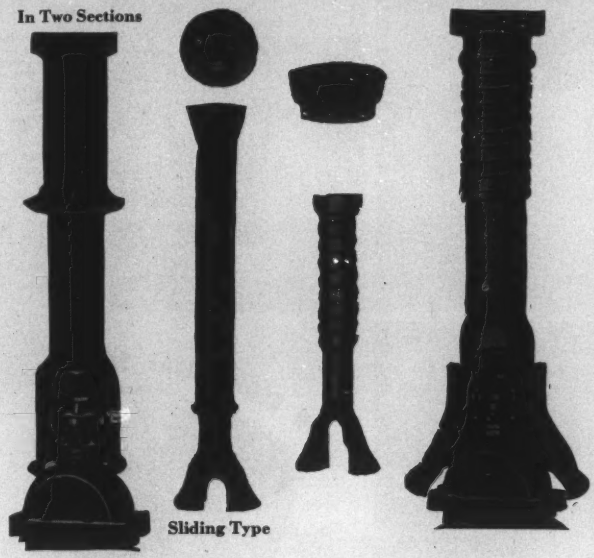
Make Your Own

CONCRETE METER BOX BARRELS *with* FORD *Singl-use Fiberforms*

Water utilities that find vitrified or concrete tile expensive or difficult to obtain, may now make their own barrels for outside meter settings with Ford Singl-use Fiberforms which are inexpensive and easy to use. Forms made for 15", 18" and 20" ID barrels and in lengths of 15", 18" and 24". Use two for deep settings. Send for full information.

FOR BETTER WATER SERVICES **FORD**

THE FORD METER BOX COMPANY, INC. Wabash, Indiana



The Three "Bigs" in Buying

1. *Standardization*
2. *Dependability*
3. *Quick Delivery*

ESTABLISHED 1849

BINGHAM & TAYLOR

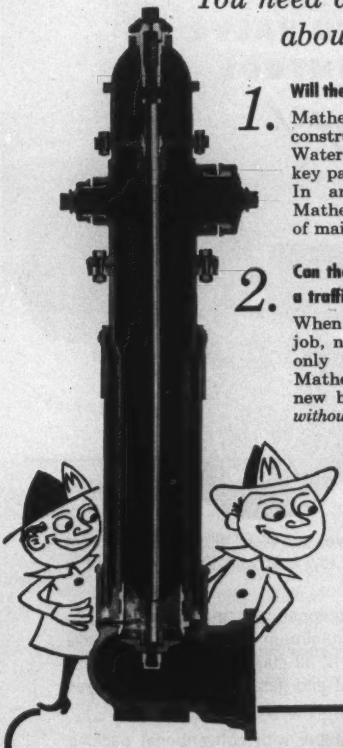
CORPORATION

Genuine Buffalo Cast Iron
Service. Valve. Roadway and Meter Boxes
Manhole Frames and Covers

CULPEPER, VIRGINIA

SPECIFY MATHEWS for DEPENDABLE PROTECTION

You need ask only two questions about fire hydrants:



1. Will they be ready for instant use in an emergency?

Mathews Hydrants are so simply and soundly constructed that they are always ready. Water, ice and sandy deposits cannot reach key parts to rust or freeze, or clog operation. In any weather and all temperatures, Mathews Hydrants function with a minimum of maintenance.

2. Can the hydrant be quickly replaced if broken in a traffic accident?

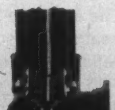
When you have Mathews Hydrants on the job, neighborhood fire protection can suffer only momentary interruption. A broken Mathews Hydrant can be replaced with a new barrel in less than half an hour, and *without excavating.*

R. D. WOOD COMPANY

Public Ledger Building, Independence Square
Philadelphia 5, Pa.

Manufacturers of "Sand-Spun" Pipe (centrifugally cast in sand molds) and R. D. Wood Gate Valves

Stuffing box plate, cast integral with nozzle section, provides a positively leakproof construction. It prevents any water or sand from reaching the operating thread. No rusting, freezing or clogging. No wear on thread through sand abrasion.



Available for mechanical joint connections

Main valve, true compression type, opens against and closes with water pressure. The higher the pressure, the tighter the valve. When hydrant is broken in traffic accident, there's no leakage with consequent dangerous loss of pressure.

MUELLER®

CUT-IN SLEEVE AND VALVE GIVES ADDED CONTROL—

at low cost!

You can add needed control valves on 4", 6", 8", 10" and 12" cast iron mains quickly, easily and at low cost with the Mueller Cut-In Sleeve and Valve.

Both the sleeve and valve are equipped with mechanical joint to give permanent connections in a minimum of time. A Cut-In Sleeve and Valve of a nominal size will fit all classes of pipe in that size range, by using a specific type of end gasket. A variety of types of end gaskets are available for all classes of pipe.

The Mueller Cut-In Valve is available with conventional packing or "O" ring stem seals. The exclusive Mueller four-point disc wedging mechanism exerts seating pressure near the perimeter of the discs – prevents disc deflection and assures positive shut-offs. Consult your Mueller W-96 Catalog, your Mueller Representative, or write direct for complete information.

MUELLER CO.

Dependable Since 1857

MAIN OFFICE & FACTORY DECATUR, ILLINOIS

Wide World Photo

this could happen in your town

THIS CITY STREET CAVED IN when a water main broke. It's a dramatic, but not an unusual example of what water does to road bedding.

Protect your roads and pipes from the problems of leakage by Centrilining. This patented process thoroughly cleans and centrifugally lines pipes with cement mortar. Leakage, tuberculation and interior corrosion are permanently checked . . . carrying capacity and distribution pressure are increased to "better than new" highs. And the whole process is done with pipes in place . . . no excavation . . . no traffic disruption.

With Centrilining you can save your water mains . . . and streets too.

Write today for free booklet

CEMENT-MORTAR LININGS FOR PIPES IN PLACE

3,500,000 FEET **OF EXPERIENCE**

CENTRILINE CORPORATION

A subsidiary of Raymond Concrete Pile Co.

140 CEDAR STREET, NEW YORK 6, N. Y.

Branch Offices in Principal Cities of United States and Latin America

Which uses the most water?

Filling Station —or Church Building?

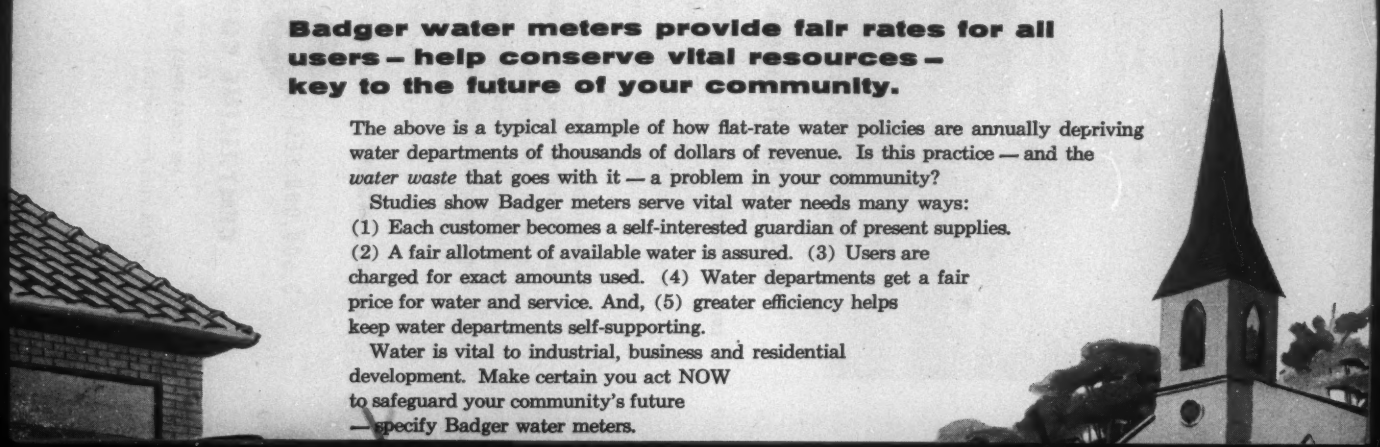
ANSWER: Before one Midwestern community installed water meters, gas stations were billed a flat rate of $12 per year; so were the churches. Last year, the average bill for all filling stations was $48, while in only one case did a church use more water than the minimum charge of $7.20.

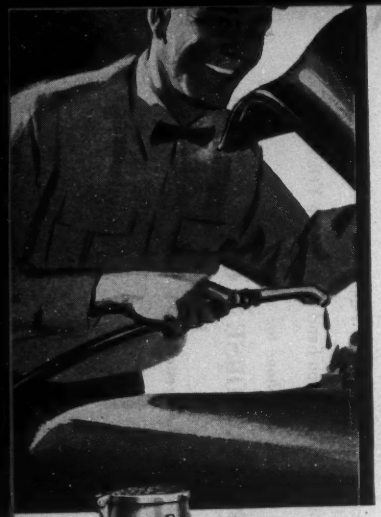
Badger water meters provide fair rates for all users – help conserve vital resources – key to the future of your community.

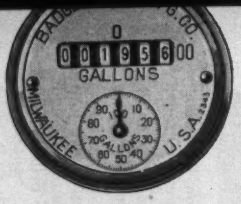
The above is a typical example of how flat-rate water policies are annually depriving water departments of thousands of dollars of revenue. Is this practice — and the *water waste* that goes with it — a problem in your community?

Studies show Badger meters serve vital water needs many ways: (1) Each customer becomes a self-interested guardian of present supplies. (2) A fair allotment of available water is assured. (3) Users are charged for exact amounts used. (4) Water departments get a fair price for water and service. And, (5) greater efficiency helps keep water departments self-supporting.

Water is vital to industrial, business and residential development. Make certain you act NOW to safeguard your community's future — specify Badger water meters.

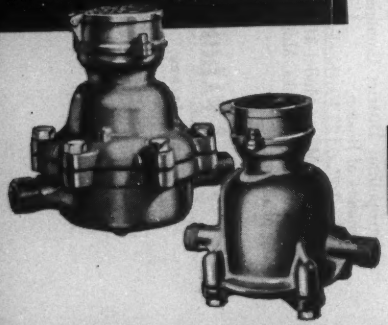






Sensitive registering key to maximum revenue per installation

One of the greatest "business machines" your community can own. Badger water meters are always on duty. Dial accurately registers all flow from the smallest stream to maximum meter capacity. Service records also prove the dependability of Badger meters . . . show why waterworks men agree, "It pays to install Badger meters."



For complete details on how Badger meters can help your community save water, work and money, write us.

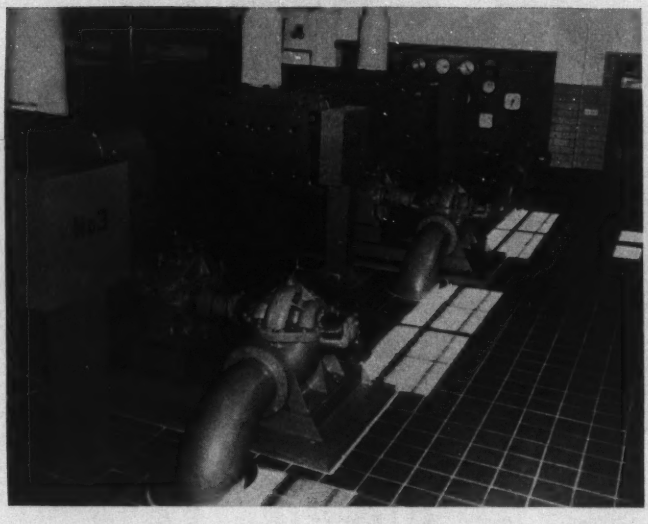
Badger Water Meters

BADGER METER MFG. CO. • MILWAUKEE 45, WISCONSIN

Water is Vital

"Badger Meters have conserved it for 50 years"

ANOTHER FAIRBANKS-MORSE LEADER



F-M Equipment at the New M.D.C. Newton Street Pumping Station, Brookline

The three Fairbanks, Morse Diesel Engine Driven Pumps pictured above are capable of delivering 4.10 Mgd under present operating conditions into the system of the M.D.C. and the town of Brookline.

Under future conditions, following completion of the Commission's proposed water system lines, the pumps will deliver water at the rate of 5.10 Mgd into the M.D.C.'s Southern Extra-High service lines and into the Bellevue Hill Reservoir.

The design and supervision of this new station for the Water Division of the Metropolitan District Commission was by Coffin & Richardson, consulting engineers of Boston.

The 10″ x 12″ Fairbanks, Morse Figure 5813 Pumps are each driven by a Fairbanks, Morse Model 31A6¼ Diesel Engine.

A trained Fairbanks, Morse Engineer is on hand to help solve YOUR pumping problems.

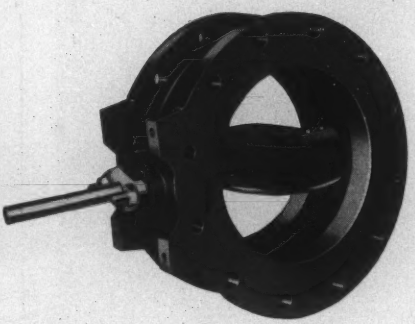
Write or telephone to:

FAIRBANKS, MORSE & CO.

178 Atlantic Avenue **Boston 10, Massachusetts**

Telephone: LAfayette 3-3600

Your source for Blue Chip Products



Builders AWWA Standard Butterfly Valves. Engineered specifically for water works service.

The New England Sales Office of B-I-F Industries offers you the most complete line of water works equipment: PROPORTIONEERS feeding and proportioning equipment for solutions — OMEGA volumetric and gravimetric feeders for dry and fluid materials — BUILDERS equipment for recording, indicating, and controlling . . . and for chlorination. We welcome the opportunity to serve you. B-I-F INDUSTRIES, INC., 291 Washington Street, Islington, Mass. Tel.: DEdham 3-5690.

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PROVIDENCE, RHODE ISLAND

B-I-F METERS FEEDERS CONTROLS

CITY OF BROCKTON — SILVER LAKE STATION

Shown above are two 400 HP motor driven pumping sets each consisting of two single stage pumps in series with overhead cross-over piping. The unit design conditions are 4500 G.P.M. against 300 Ft. T.D.H. The firm of Hayden, Harding and Buchanan prepared the plans and specifications and supervised the installation of all the equipment.

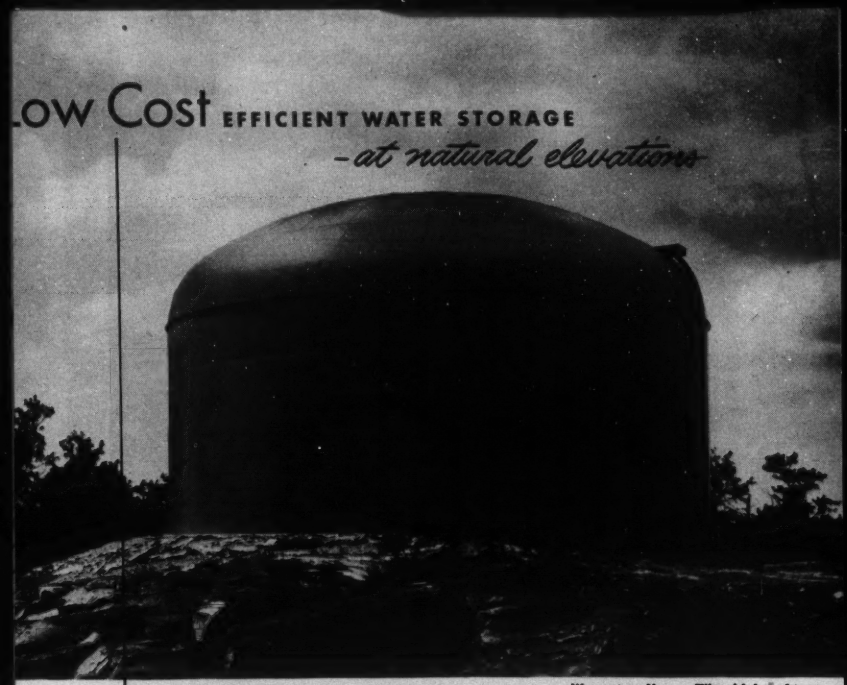
TURBINE EQUIPMENT COMPANY OF NEW ENGLAND

80 Federal Street **Phone LIberty 2-5993** **Boston 10, Mass.**

New England Representatives for

DE LAVAL STEAM TURBINE COMPANY

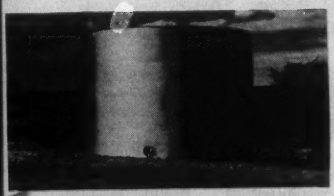
TRENTON, NEW JERSEY



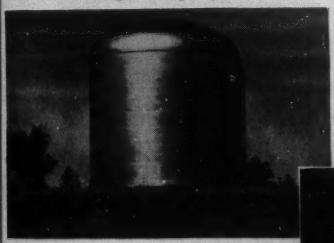
Worcester, Mass.—Ellipsoidal roof type, 1,500,000 gallons capacity.

* Steel Reservoirs

by PITTSBURGH • DES MOINES



Liberty, N.Y.—Flat roof type, 300,000 gallons capacity.



St. Paul, Minn.—Ellipsoidal roof type, 2,000,000 gallons capacity.

The advantages of strength, simplicity, and economy of construction make Pittsburgh-Des Moines Steel Reservoirs your outstanding choice for water storage where elevated sites are available.

Durable for a lifetime, guaranteed in design, workmanship and material, P-DM Steel Reservoirs are built in flat-, dome-, or ellipsoidal-roof types, unornamented, or with pilasters and other architectural features as specified. *Write for our latest descriptive Steel Reservoir Bulletin.*

PITTSBURGH • DES MOINES STEEL CO.

Plants at PITTSBURGH, DES MOINES and SANTA CLARA

Sales Offices at:

PITTSBURGH (25). . . . 3424 Neville Island	DES MOINES (8). . . . . 925 Tuttle Street
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CHICAGO (3). 1228 First National Bank Bldg.	SEATTLE . . . . . 532 Lane Street
LOS ANGELES (48). . . 6399 Wilshire Blvd.	SANTA CLARA, CAL. . . . 631 Alviso Road

BIG valve... little space

The disc in this 84-in. low pressure Butterfly Valve must withstand 138,000 lbs. of force when closed against a 25 P.S.I. line pressure . . . yet the face to face dimension is only 18 in. and the entire valve weighs just 12,000 lbs. Even under this severe loading Pratt design keeps the bearing pressure well below 2500 P.S.I.

Pratt Rubber Seat Butterfly Valves are manufactured in sizes from 10 in. to 168 in. in diameter, in pressure ratings up to 125 P.S.I. Their low initial cost, easy installation and proven low maintenance make them the logical solution to valving problems, both for new projects and as replacements.

WRITE FOR FULL DATA:
HIGH PRESSURE VALVES, BULLETIN 4B
LOW PRESSURE VALVES, BULLETIN 1B
"UNDERSTANDING THE BUTTERFLY VALVE", BULLETIN 2B

HENRY PRATT

RUBBER SEAT Butterfly Valves

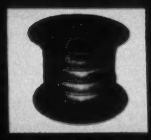
Henry Pratt Company, 2222 S. Halsted St., Chicago 8, Ill. Representatives in principal cities

THE NATION'S No. 1 SOURCE

for dependable pipe line equipment

Like hundreds of Pollard customers, you'll find it mighty pleasant and convenient to order pipe line equipment from Pollard.

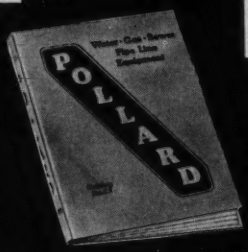
Here, under one roof and listed in one comprehensive catalog, are all the time and money saving products you need on the job. Each is guaranteed for dependability, economy. Each is backed by Pollard service.



MANHOLE CUSHION

DIAPHRAGM PUMPS

PIPE PUSHER



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JOSEPH G. POLLARD CO., INC.

PIPE LINE EQUIPMENT

PIPE LINE EQUIPMENT

Place your next order with POLLARD

If it's from POLLARD . . . it's the Best in Pipe Line Equipment

NEW HYDE PARK • NEW YORK

Branch Offices: 1064 Peoples Gas Building, Chicago, Illinois
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SINCE 1861

LUDLOW

How the consolidation of **two great valve companies** can help you

In January, 1955, two great valve companies consolidated their resources. They had been neighbors in Troy, New York and friendly competitors for two thirds of a Century. The names remain the same but Rensselaer became a Division of The Ludlow Valve Manufacturing Company—All now under one management in one plant, with unit responsibility to its customers.

THE BEST IN DESIGN

You, the Water Works Executives are the final judges. You have cast your ballots in the form of extensive purchases over the last half century. In many cases the result is obvious, but your opinions on A.W.W.A. valves and hydrants vary. Therefor both lines will be continued and any Representative of the combined companies can show you the characteristics of each. In the meantime our search for ever-better valves goes on.

THE BEST IN PRODUCTION

The production facilities of the combined Companies have been consolidated and at the same time are being greatly expanded. Only the newest, most modern equipment has been retained. Hundreds of thousands of dollars worth of new equipment is expanding the capacity and giving even greater assurance of our ability to serve. This means insurance for you, more local stocks to draw on, and shorter deliveries on heavy equipment. This all means lower inventories for you.

THE LUDLOW VALVE MFG. CO., INC. TROY, N.Y.

10

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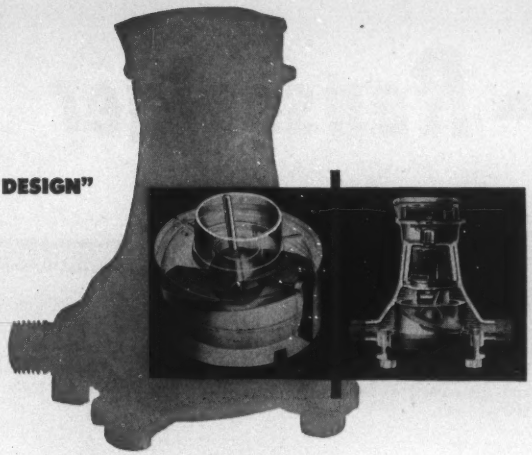

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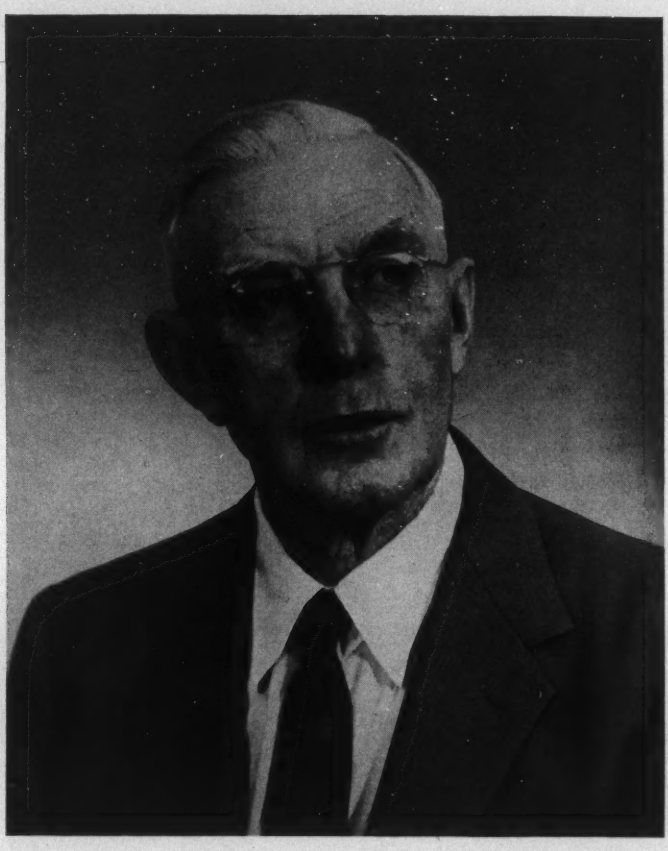

J. Arthur Jura

ALLAN F. MCALARY

President of the New England Water Works Association
1955-1956

New England Water Works Association
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MICRO-STRAINING AND ITS APPLICATIONS

BY P. L. BOUCHER*

[*Read March 17, 1955.*]

INTRODUCTION

After the end of the last world war, interest was developed in Britain in the possibilities of adapting the rotary-drum strainer or screen, hitherto used only for the elimination of gróss suspended solids from water, as a filter capable of removing microscopic solids, with performance equal to that of rapid-sand filters operated without co-agulants. Much success was achieved and the process of Micro-straining came into being, with the subsequent establishment of a wide field of applications in water purification.

Today there are nearly sixty Micro-straining installations either in operation or in course of construction, with individual flow capacities ranging from a few hundred-thousand gallons to over 100 m.g.d. The largest user, at present, is the Metropolitan Water Board, London's water supply authority, who have in service, or under construction, four plants with a total capacity of 200 m.g.d. more than half of London's daily water consumption.

Other major British water undertakings employing the process on a full scale include those of the cities of Bristol, Liverpool, Edinburgh, Birmingham, Norwich, Belfast and Preston, and the Colne Valley Water Company, whilst dozens of smaller undertakings, both municipal and industrial, have followed suit.

The applications include primary filtration prior to rapid and to slow-sand filtration, Micro-straining as a sole filtration process, and even as a special "polishing" process after rapid and slow-sand filtration. The final clarification of sewage effluent is another important

*Chief Civil Engineer, Glenfield & Kennedy, Ltd., 105 Park St., London, W. 1, England; Vice-President, Glenfield & Kennedy, Inc., 677 5th Ave., New York, N. Y.

municipal employment, whilst similar applications are found in industrial water supply and effluent treatment.

The advantages enjoyed by users of Micro-straining plant are greatly reduced capital charges, nominal operating costs, small compass of installations, overall hydraulic head loss reduced to inches with elimination of pumping in most cases, automatic operation and very low maintenance. In Britain and elsewhere, the adoption of Micro-straining in cases where conventional filtration plant would otherwise have been employed, has achieved important economies in the expenditure of public funds.

In this Paper it is proposed to describe the equipment, to give some account of selected samples of installations, and to conclude by outlining the problems of design. It is hoped that American Waterworks men may find points of interest in this development which, in the last decade, has become an established part of British waterworks practice.

Equipment

The Micro-strainer is a rotary drum filter which operates in gravity flow conditions, usually installed in reinforced concrete tankwork. One end of the drum is open and runs against a matching inlet opening in a Meehanite end frame built into a wall which divides the filtered from the unfiltered water. The other end of the drum rotates against a similar end frame which is blanked off and stands in the filtered water compartment. The filtering medium is fixed to the drum periphery and the flow of water is therefore axially into the drum and radially out of it through the filtering medium, intercepted solids being retained on the inner drum surface. The flow pattern is illustrated by Fig. 1. The upper portion of the drum is above the water level and external backwashing jets span the drum length in such a way that the retained solids are flushed continuously, as the drum rotates, into an internal waste-water hopper, the lip of which is also above the water level. A hollow axle on which the waste-water hopper is mounted allows egress of the waste water from the drum.

Mechanically the equipment is extremely simple but of high quality construction, in conformity with best waterworks practice. The drum cage is of bronze, the filtering medium of stainless steel. The drum rotates on specially manufactured stainless-steel ball and roller

bearings carried by the central hollow shaft, which is mounted between the two massive end frames. Running clearances are doubly sealed to ensure absence of leakage of unfiltered water.

The drum is rotated by means of a large diameter spur-ring of

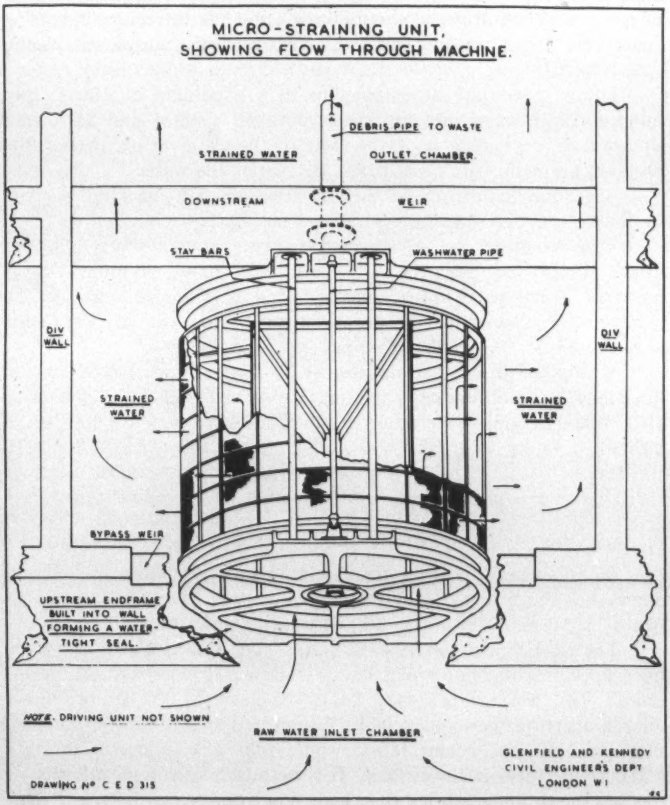


FIG. 1.—DIAGRAM OF FLOW PATTERN THROUGH MICRO-STRAINER.

bronze, driven through gearing and speed-change mechanism by a small-power electric motor.

Special, self-cleaning, adjustable, wash-water jet nozzles of bronze and stainless-steel construction keep wash water consumption, which depends upon the quality of the raw water and the necessary speed of rotation of the drum, at the minimum possible in various operating conditions. The washing area is enclosed by transparent plastic splash guards.

Drum speeds are adjustable up to a maximum of 100 ft. per minute. Wash-water pressure varies between about 5 and 25 lb. per square inch, depending upon the state of the water being filtered, the pressure normally being the higher the dirtier the water.

A general impression of the construction of a machine is given by the perspective drawing reproduced in Fig. 2.

The machines are constructed in four standard sizes having drums respectively 2′6″, 5′0″, 7′6″ and 10′0″ diameter. They cover unit flow capacities of up to about 12 m.g.d. U.S., depending upon the quality of the raw water. Even for the largest unit, the driving motor is rated only at 1¼ B.H.P., for normal drum speeds.

Multiple-unit installations are used for large flows and to provide flexibility, and any flow capacity can be obtained from 7′6″ diameter or 10′0″ diameter units operating in parallel. Machines are located in individual tanks with inlet and outlet control valves or sluices. In the largest plants it is appropriate to keep constructional costs at their lowest by installing pairs of machines in individual tanks.

The Filtering Media

Micro-straining fabrics are available with a number of different degrees of fineness. The most commonly used are three, known as Mark II, Mark I and Mark 0, in descending order of aperture size.

The fabrics are woven of stainless-steel and in the order given have apertures of approximately 65 microns, 35 microns and 23 microns. The finest fabric, the Mark 0, has 144,000 twenty-three-micron apertures per square inch. A standard 10′0″ diameter machine fitted with this particular fabric would thus have about 7,000,000,000,000 apertures in its surface. It is because of the high porosity of the Micro-straining fabrics that high flow ratings can be used, com-

pared with sand-filtration, under extremely low differential pressures, and therein lie the economies to be achieved by the employment of the process.

Fig. 3 shows the construction of Micro-straining fabric, and illustrates the definitions of the "inner" and "outer" limiting apertures.

The fabric is fitted to the drums in contact with coarse mesh stainless-steel fabric which provides mechanical support and also has the effect of breaking down surface-tension which would otherwise

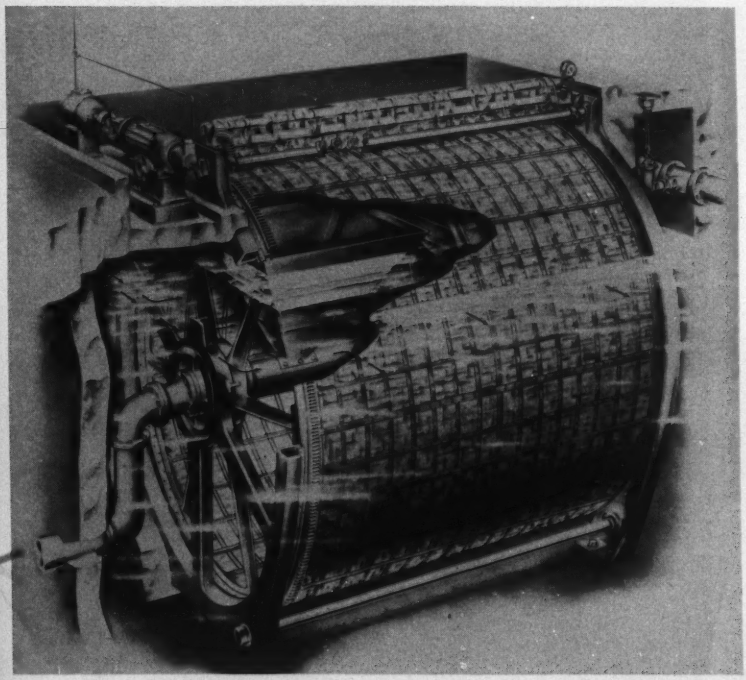


FIG. 2.—PERSPECTIVE CUT-AWAY DRAWING OF LARGE-CAPACITY MICRO-STRAINING UNIT.

make it impossible for low-pressure back-washing arrangements to be used. This coarse fabric has, therefore, become known as "pilot" fabric.

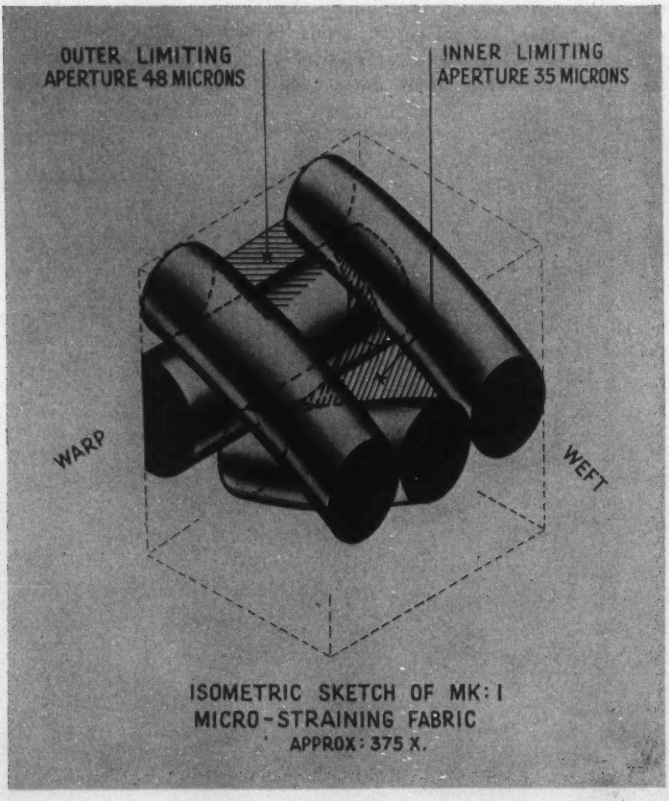


FIG. 3.

Allowable flow ratings through the fabrics vary from about 2,000 gallons U.S. per-square-foot-per-hour down to about 700 gallons U.S. per-square-foot-per-hour, depending upon the concentration of solids in the raw water. Arbitrary flow ratings cannot be adopted in designing Micro-straining installations and the methods employed involve the conception of the filtrability of fluids, dealt with briefly towards the end of this Paper.

The efficiency of filtration by Micro-straining depends upon the formation of a thin mat of intercepted solids on the inner surface of the filtering fabric. The fabric, of course, positively intercepts solids larger than the apertures, and a mat is formed which also intercepts a large proportion of solids smaller than the apertures. High filtration efficiencies are therefore obtained, even at the elevated flow ratings quoted above. The mat is very thin compared with the mat, or *schmutzdecke,* formed on a sand bed, and it is washed off at the speed of rotation of the drum, to be rapidly reformed as the cleaned fabric re-enters the water. The head loss rarely exceeds four or five inches, even at the highest flow ratings, and the intercepted solids never therefore become so consolidated that they cannot be removed by the low-pressure water jets.

Although microscopic suspended matter of all kinds is filtered out, no appreciable reduction of true colloidal matter, or of colour in solution can be expected by Micro-straining, unless coagulants are employed in prior treatment.

Some Applications of Micro-straining

Installations of the Metropolitan Water Board

The first installation for London's Metropolitan Water Board was at Surbiton, in the Thames Valley Group of Works, where a 3 m.g.d. experimental unit was put into operation early in 1946, upstream of a large slow-sand filter (No. 6) and controlled by observation of two other slow-sand filters (Nos. 7 & 8) operating in parallel. All filters were dealing with stored River Thames Water, very liable to Plankton invasions, Bed No. 6 having pre-treatment by micro-straining, Beds Nos. 7 & 8 taking the raw water. From February, 1946, to March, 1950, Bed No. 6 passed about twice as much water

between cleaning operations as Beds Nos. 7 and 8, at a rate of filtration nearly 1.7 times as high.[1]

Kempton Park

These results were so promising that by March, 1948, a much larger installation was started up at Kempton Park Works, also in the Thames Valley where a rapid-sand pre-filtration plant already existed. One ¾-acre slow-sand filter (No. 1) was isolated to receive Micro-strained water, and two other beds (Nos. 4 & 5) of the total number of 12, were selected for comparative observation.

Typical results published[2] up to March, 1950, showed that for equal rates of filtration through beds Nos. 1, 4 and 5, the quantity filtered per acre of slow-sand filter beds cleaned was, for the primary sand filtered water, 198,000,000 gallons, and for the Micro-strained water 187,000,000 gallons.

The Kempton Park installation consists of four 7'6" diameter x 5'0" wide machines, fitted with Mark I Micro-straining fabric (then the finest available), and it proved to have a flow capacity in normal water conditions of approximately 15 m.g.d. Wash-water consumption, at this rate of flow, was little more than 1% of the volume filtered. The installation, which is still in operation, is illustrated by Fig. 4.

The total cost of the Micro-plant, including building and main laying, was (in 1948) £12,795.0.0d. ($35,826.00). The space occupied in plan measures 44 ft. x 14 ft. and the hydraulic loss across the installation is normally below 6 ins.

The rapid-sand filtration plant, completed in 1929, with which the Micro-straining installation was compared, consists of 24 Paterson Filters, each of 416 sq. ft. area, with 2 ft. 3 ins. of 0.78 millimetres sand of uniformity co-efficient 1.57, operating at 200 gallons per square foot per hour. The sand was renewed in 1947, and the beds were operating at high efficiency and passing together up to 60 m.g.d.

Lea Bridge

The Board's Lea Valley Group of Works was chosen for the next installation of Micro-straining plant, this time for a flow of 75

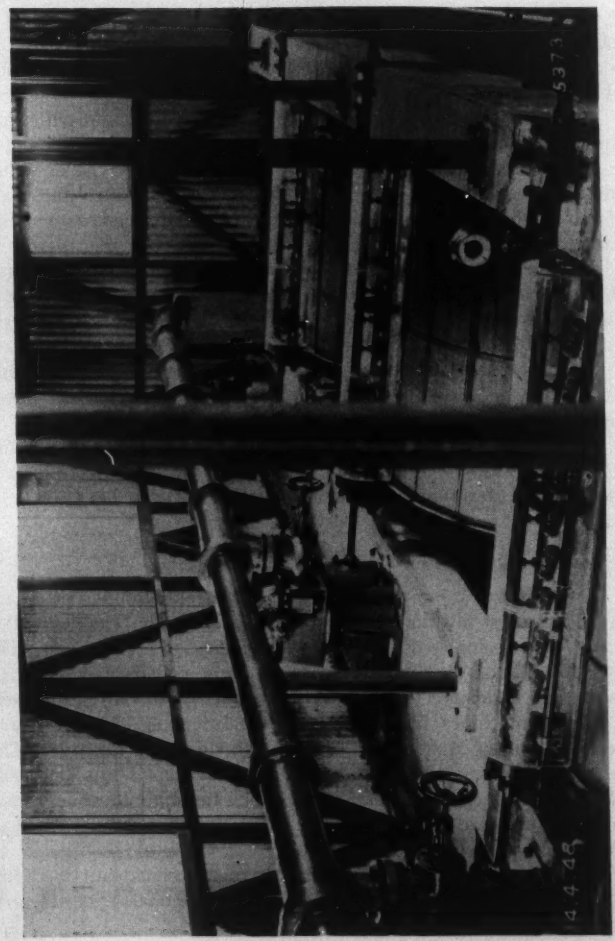


FIG. 4.—MICRO-STRAINERS AT KEMPTON PARK WORKS, LONDON, ENGLAND.

m.g.d. A decision to employ Micro-straining was taken in 1949. The plant, which was put into operation at the end of 1954, consists of ten of the largest units, each having drum 10′0″ diameter x 10′0″ wide, and again fitted with Mark I fabric.

Each pair of units has a separate tank with inlet and outlet isolating sluices, and is generally arranged as shown by Figs. 5, 6 and 7

Flow through the installation is by gravity from the aqueduct leading in stored River Lea Water, which flows through the plant and back into the aqueduct, the raw and Micro-strained water being separated by a steel-pile dam. The pre-filtered water then passes to the slow-sand filters without any hydraulic change other than a few inches of head loss across the dam. It need hardly be stated that if sand-filtration had been employed, low-lift pumps of 75 m.g.d capacity would have been required.

No results have yet been published of the operation of this plant, as it has only recently been put into full operation, but it has saved a very large capital expenditure on additional sand filters.

The cost of the Micro-strainers (1949) was £62,000.0.0. ($173,-600) and the total overall cost, including all works involved, was given by the Metropolitan Water Board (1954) as £115,000.0.0d. ($322,000.)

Ashford Common Works

The Ashford Common Works of the Metropolitan Water Board is an entirely new filtration and pumping station in the Thames Valley not yet fully completed, for a flow capacity of 112 m.g.d. The largest installation of Micro-strainers yet to be constructed is, at the present time under erection at this site. Twenty-four 10′0″ diameter x 10′0″ wide machines, fitted with Mark I fabric, will deal with stored water from Queen Mary Reservoir, prior to slow-sand filtration. The general arrangement of the Micro-straining section of this station is shown by Fig. 8, the main features of which resemble those of a conventional rapid-sand filtration plant, with pairs of Micro-strainers in each tank instead of sand beds.

The estimated total cost of this Micro-straining plant is £330,-000.0.0d. ($924,000) with about £200,000 ($560,000) represented by the Micro-strainers.

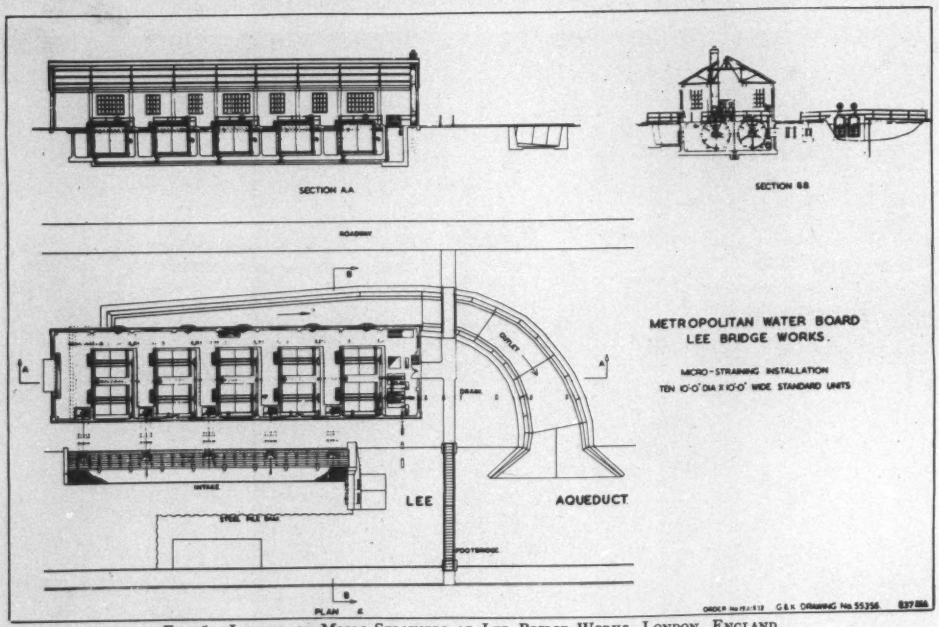


FIG. 5.—LAYOUT OF MICRO-STRAINERS AT LEE BRIDGE WORKS, LONDON, ENGLAND.

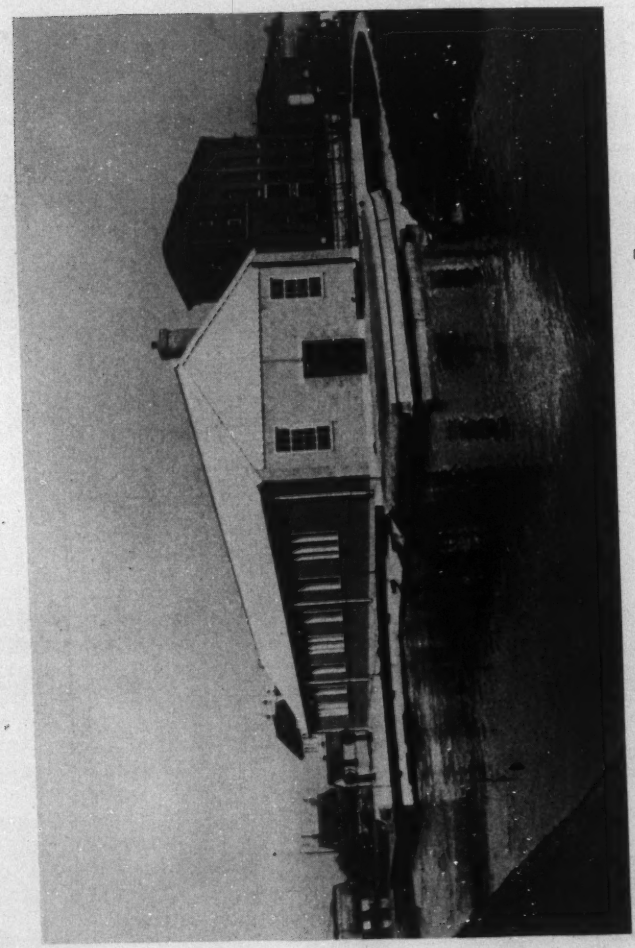


FIG. 6.—MICRO-STRAINER HOUSE, LEE BRIDGE WORKS, LONDON, ENGLAND.

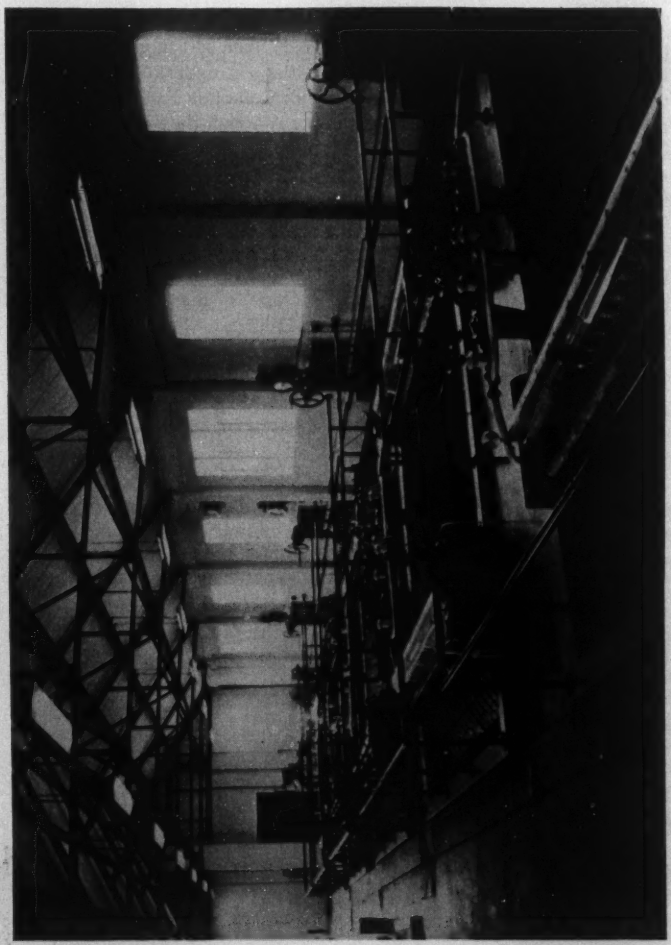


FIG. 7.—INTERIOR OF MICRO-STRAINER HOUSE, LEE BRIDGE WORKS, LONDON, ENGLAND.

The total flow capacity of the Micro-straining installations supplied to the Metropolitan Water Board is over 200 m.g.d. more than half the water supplied by the World's largest undertaking. It is interesting to note that the Surbiton and Kempton Park installations are still in continuous service operation, although they were originally installed purely for experimental purposes.

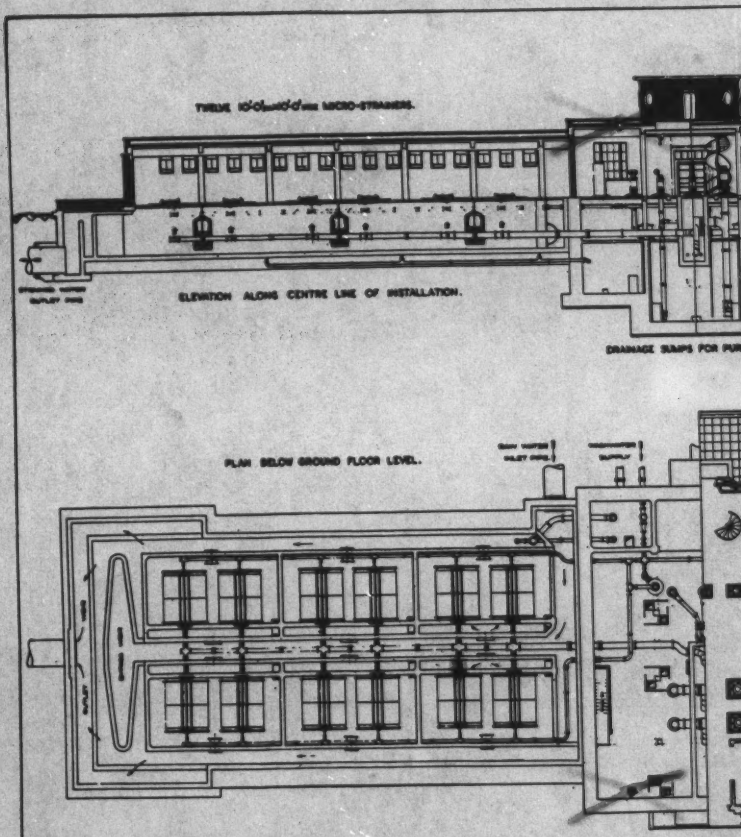
Bristol Waterworks Company

A few months after the first Micro-strainer was started in London, at Surbiton Works, the first of three installations for the Bristol Waterworks Company was commissioned. A year later, experience with this initial plant was published.[3] Two 7'6" diameter x 5'0" wide machines were employed to deal with 6.25 m.g.d. of water impounded in Barrow Reservoirs or pumped from the Company's Cheddar source, and the object was to provide primary filtration in order to double the flow capacity of two of the slow-sand filters to meet an increasing demand for water, without the necessity of constructing additional sand filters.

This first installation was completely successful and it was demonstrated during 1947 that the gallons filtered per square yard of slow-sand filter cleaned, for the two beds taking Micro-strained water were 31,286, compared with 12,897 gallons for the then remaining beds, 10, taking unstrained water. Moreover, the bacteriological results from the filters taking strained water were noted to be somewhat better than those from the filters taking the raw water.

The installation of this first plant, providing pre-filtration to two only of the 12 slow-sand filters, put up the capacity of the whole station by 15% at a total cost (1946) of £11,548.0.0d. ($32,334.40), including considerable alteration to mains. For 1947, the total operating cost of the Micro-straining plant, including maintenance, power, repairs, and wash-water (priced at 6d. per 1,000 gallons Imp.) was given as £–.8.0d. ($1.12) per million gallons (1.25 m.g.d.U.S.)

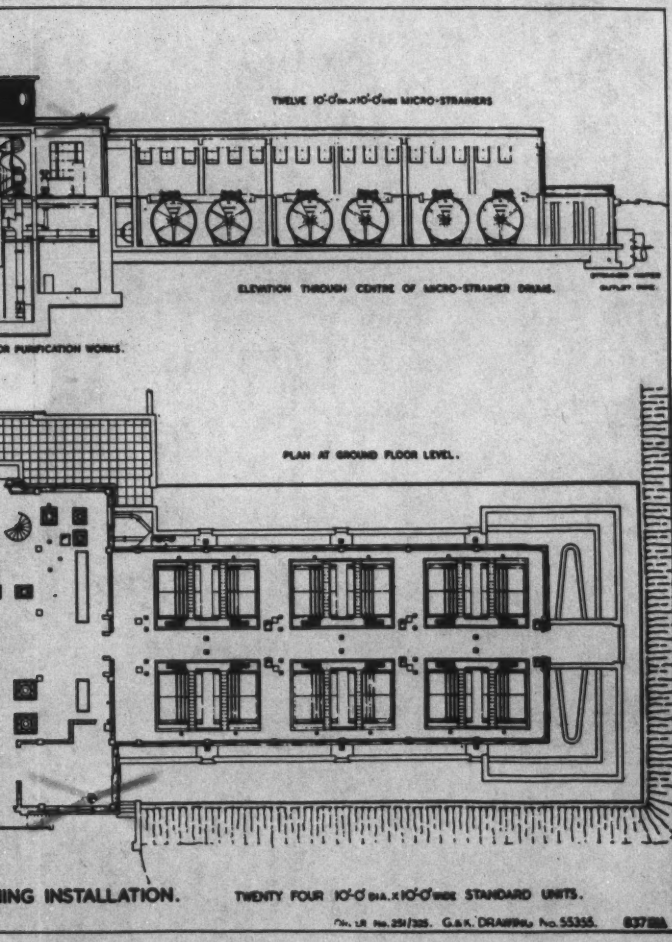
The first Barrow Plant is still in operation with its original Mark I fabric, no general fabric replacements having been necessary in eight years. As the demand for water increased at Bristol, two further Micro-straining installations were added, and today, the three



METROPOLITAN WATER BOARD.

ASHFORD COMMON WORKS. MICRO-STRAINING

FIG. 8.


TWELVE 10'-0" DIA. x 10'-0" WIDE MICRO-STRAINERS
ELEVATION THROUGH CENTRE OF MICRO-STRAINER DRUMS.
OUTLET MAIN.
OR PURIFICATION WORKS.
PLAN AT GROUND FLOOR LEVEL.
HING INSTALLATION.
TWENTY FOUR 10'-0" DIA. x 10'-0" WIDE STANDARD UNITS.
No. LR No. 251/325. G.& K. DRAWING No. 55355.

8.

plants deal with 22 m.g.d. of water, and have made unnecessary the costly construction of at least seven additional sand beds.

Wash-water consumption for a typical year (1953) was reported as 1.32% of the flow filtered. Running costs have remained very low, labour charges, in particular, being reported for the whole operational period as negligible.

City of Edinburgh

The capital city of Scotland decided, at an early stage of the development of Micro-straining, to employ the process also for the primary filtration of impounded water prior to slow-sand filtration. The first installation was for a new purification station serving an outlying area known as Rosebery in the County of Midlothian, with a supply of up to 7.5 m.g.d. This plant went into operation early in 1953, and has run successfully ever since.

The second, and larger, plant went into service in January of the present year. It is at Fairmilehead Filtration Works, which form the principal supply centre to the western half of the City of Edinburgh, and deals with nearly 20 m.g.d. of water from Tala. This flow will eventually be increased to about 26 m.g.d.

The flow rating of the existing filters was originally only 13 m.g.d. but this has now been raised to the 20 m.g.d., and will eventually reach the 26 m.g.d. without any further filter construction.

The Micro-plant consists of six 7′6″ diameter x 5′0″ wide drums, fitted with Mark I fabric, with provision for the future installation of two more similar drums. The machines are arranged in pairs in separate chambers with inlet and outlet isolating sluices.

The raw water entering Fairmilehead Works has a residual head of about 100 ft. so that it was possible to install in the scheme a hydro-electric generating plant to produce some 80 Kw. from about 8 m.g.d. of water. This power is sufficent to provide for the operation of the Micro-strainers and wash-water pumps, to heat and light the building, and to provide for the Water Department's other requirements at Fairmilehead.

George Baxter, City Water Engineer of Edinburgh, gave estimates[4] of the cost of various methods of increasing the capacity of Fairmilehead Filter Station to the extent required. To have provided

for a 50% increase by the construction of additional slow-sand filters would have cost (in 1951) £88,000.0.0d. ($246,400). Primary rapid-sand filters to pre-filter the water before passing it to the existing slow-sand beds would have cost about £112,000.0.0d. ($313,600). The cost of obtaining the 50% increase by adopting Micro-straining was then established at about £40,000.0.0d. ($112,000) with a saving of some £48,000.0.0d. ($134,400) against the cost of additional slow-sand filters, and at least £72,000.0.0d. ($201,600) against the adoption of primary sand filtration.

The actual total cost of the Micro-straining plant at January, 1955, taking into account increased costs of labour and materials since the original estimates were made in 1951, was £64,000.0.0d. ($179,200), including all accessories.

Colne Valley Water Company

The plants so far quoted are selected from a score of installations where slow-sand filtration is used, the Micro-plants forming the pre-treatment stage. The Micro-strainers for the Colne Valley Water Company are also pre-filters but, in this case, will operate upstream of treatment works in which rapid-sand filtration is employed.

The City Lane Pumping, Treatment and Filtration Works are constructed for a maximum flow capacity of 30 m.g.d. of water from five different sources, including three wells in the chalk and Hilfield Park Reservoir which, at certain seasons of the year, impounds well water for reserve use.

Lime softening is expected to be employed, with a coagulant, the equipment of the main purification works including three 75 ft. diameter Accelators, and twelve 1,080 sq. ft. rapid-sand filters with arrangements for the application of CO_2 for alkalinity neutralization and chlorine for final sterilization.

The Micro-straining installation is located between this treatment plant and the Hilfield Reservoir, the water in which, because of its chemical composition and because, also, of the characteristics of the reservoir, is expected to suffer heavy seasonal infestations of Plankton. Six 7′6″ diameter x 5′0″ wide units, fitted with Mark I fabric, will deal with 15 m.g.d. of water from this reservoir, and the operation of this Micro plant will ensure that the final purification

works will receive a water of much more uniform character than would otherwise be the case, and that overloading of the Accelators and filters during Plankton peaks will not occur.

The Micro-plant is, at present, under erection at the site, and is expected to go into service later in the year.

A number of similar applications have been made of Micro-straining plants, both in Britain and elsewhere.

Borough of Preston Water Department

There are many water supplies which, in the raw state, are clear and uncoloured, and carry only a sparse population of living organisms and microscopic debris, being otherwise fit for distribution after sterilization. Micro-straining, in such cases, is suitable as a sole filtration process, permitting very great capital economy to be effected.

In March 1953, the Town Council of Preston confirmed a resolution of the Water Committee that a Micro-straining Plant, for the treatment of the whole water supply from the Town's Alston Reservoirs, should be installed, together with ancillary chemical plant, for a flow of nearly 20 m.g.d., the whole water supply of the Town. This decision was taken after some two years' operation of a prototype Micro-strainer, installed on the Town's water supply for dealing with 2½ m.g.d. supplied to Courtauld's Rayon Factory. The Engineer has stated in his report to the Town Council that this plant has given complete satisfaction, and that Micro-straining will suffice in the case of the Town's water supply, to produce water of excellent quality for a capital investment of less than half that of sand-filtration plant.

The installation will comprise eight electrically-operated Micro-straining drums, each 7′6″ diameter x 5′0″ wide, fitted with Mark 0 Micro-straining fabric which, in view of the nature of the raw water, is expected to give a perfectly limpid and brilliant filtrate, which will need only liming for pH correction and chlorination for sterilization.

Northampton Municipal Swimming Pool

The town of Northampton, in England, operates an open-air swimming pool, known as Midsummer-Meadow Pool, the water supply for which comes from the condenser cooling system of an adjacent

central electric generating station, this station drawing its water originally from the River Nene. The swimming pool is, in fact, supported by the purge water from the cooling water system of the power station and, consequently, has some contamination from the original river, from algal growths in the condenser cooling system, and from precipitation of dissolved solids.

The quality of the water having recently been established as unsatisfactory for bathing purposes, a decision was taken early in 1955 to employ Mark O Micro-straining, with break-point chlorination, to ensure the physical and bacteriological qualities of the water, and it is expected to have this plant in operation by the time the pool opens to the public again in May of this year. This particular application of Micro-straining is of interest in that it is the first in swimming-bath practice, and the decision to employ it at Northampton has resulted in a very considerable saving of capital cost, over what would have been necessary had the conventional sand-filtration system been adopted.*

Industrial Water-Supply Applications

There are several industrial water supply projects in which Micro-straining is employed, in cases where Municipal Water undertakings have been responsible for these supplies. Apart from these cases there are a number of others in which the industrial concern involved have put down Micro-straining installations in place of sand filtration, to prepare process water for plant use.

Industrial effluents can also be dealt with by Micro-straining, and these applications are of interest to the Water Engineer in that they can often be applied to prevent the pollution of rivers and other water courses. Some successful applications, both for process water and effluents, have been made in the Paper Industry in Britain, and these applications, in particular, have created very considerable interest in the Paper towns of the U.S.A. The problems of clarification of heavily contaminated process waters and effluents, however, are often so difficult of solution, that no single process can be employed, and Micro-straining may find its application in the final or polishing stage, after pre-treatment by other types of plant.

*At the time of going to press it is reported that the installation was entirely successful during the 1955 summer season.

Sewage Effluent Filtration

A Paper on Micro-straining would not be complete without some reference to the application of the process to sewage purification. This subject is sometimes of interest to Water Engineers in that sewage effluents are discharged to water courses and even become, eventually, part of the raw water with which the Water Engineer has to deal.

The application of Micro-straining is made after biological purification of sewage, when the problem is to reduce the suspended solids, or humus, content of the effluent, to acceptable figures. It is perfectly practicable to take effluents containing 50, 60, or even 100 p.p.m. of humus, and to reduce this humus to below 10 p.p.m. with corresponding reductions in B.O.D. by Micro-straining, employing either Mark I or Mark 0 fabrics.

The first installation in Britain was at Luton Sewage Works, where after pilot plant tests, supervised by the Department of Scientific and Industrial Research (Water Pollution Research Laboratory), a full scale installation of three 7'6" diameter x 5'0" wide machines was put in to deal with 4 m.g.d. of effluent, which was discharging into the River Lea. This plant has operated now for some five years, and has been followed by a number of other installations. Results have been published elsewhere.[6, 7]

The New Town of Bracknell, a satellite town to London, installing a completely new sewage purification plant, has commenced the installation of the first stage of a six-unit Micro-straining plant, to deal with 8 m.g.d. of sewage effluent.[8] A similar installation, for the Moe Sewage Authority in Australia, will deal with 1 m.g.d. and a full scale experimental installation, for the City of Pretoria, in South Africa, has been in operation for some while.

Design Problems

In fixing the area of a rapid-sand filtration plant, it is usually sufficient to divide the total rate of flow by a unit rate of flow known to be suitable, in order to obtain the total filter area, from which can be worked out a convenient size and number of beds. Back-washing of such filters is carried out at intervals, which are normally relatively long, and not necessarily varying with the conditions of the

Summary of Principal Installations of Micro-straining Plant

		Flow m.g.d.
Primary Filtration prior to Slow-Sand Filtration		
Metropolitan Water Board, London	4 installations	200
City of Liverpool	2 installations	70
City of Edinburgh	2 installations	22
City of Bristol	3 installations	22
Airdrie & Coatbridge Water Board		5
Tynemouth Corporation		3
Burgh of Kilmarnock		2
City of Aberdeen		3
Bridgwater Corporation		2
Primary Filtration prior to Rapid-Sand Filtration		
City of Birmingham		7
Kampala Water Supply, Uganda		4
Colne Valley Water Company		15
Filtration after Rapid and Slow Sand Filtration		
City of Norwich		12
Sole Filtration of Public Works Supplies		
Preston Corporation Water Department		20
Rio de Janiero Water Department		5.5
Belfast & District Water Commissioners		3
Dunbarton County Council		3
Morogoro Municipality		1
Auckland City Council		2
Peterhead Burgh Council		4
Larne Rural District Council		½
Ballymoney Rural District Council		¾
Burgh of Brechin		¾
Newton Stewart Water Supply		½
Inverness County Council		1/3
Sole Filtration of Industrial Supply and Effluents		
Preston Corporation, for Courtaulds Rayon Factory		3
Dillingen Steel Works, Saar		3
Consett Iron Works		4
Vernon & Co., Textile Mills		1
Co-operative Wholesale Society		1
Turner Bros. Asbestos Co., Ltd.		1
A. E. Reed & Co., Ltd. Paper Mills		3
Colthrop Board & Paper Mills, Ltd.		4
West Hampshire Water Board, for Esso Refinery		5
British Aluminum Co., Ltd.		½
Purification of Swimming Bath Water		
Borough of Northampton		1

water since, quite often, a fixed routine of back-washing is established for operational convenience.

There is certainly no such simple rule for calculating the size of Micro-straining Plant. The mat of intercepted solids retained on the fabric is built up very rapidly and is continuously washed off at the same speed. A standard 7'6" diameter x 5'0" wide machine, for example, dealing with water heavily charged with suspended solids, might have the whole of the fabric matted and back-washed four times every minute. With this type of plant, therefore, a major design problem is presented because the size of an installation for a given rate of flow depends upon the maximum rate at which the water will block the fabric. This rate is variable, and it is therefore necessary to design for a maximum concentration of solids in the raw water.

Before it was possible for Micro-straining to be developed in large scale applications, it was necessary to determine means of calculation of flow capacity in all conditions, and this necessitated the study of the laws of filtration through membranes. The conception of the filtrability of fluids provided the solution of the problem, and the fundamental work leading to this solution has been published.[9] For the present purpose it is sufficient to set down the conclusions.

The general problem, given the filtering medium to be employed and representative samples of the water to be filtered, in its worst condition as regards suspended solids, is to determine the dimensions of the plant required, the size and number of machines, and their maximum speed of operation, to pass the required flow of water under a head differential such that "break-through" of the mat will not occur.

An experienced designer, with extensive records of values of filtrability encountered with various types of raw waters, might dispense with prolonged sampling. Where time permits, however, sampling should cover all seasonal variations of the raw water, especially where very large plants are involved, as relatively small differences of the filtrability can make appreciable changes in the size and cost of an installation.

It is necessary to know the maximum value of the Filtrability Index "I" for the water to be filtered, on the filtering medium to be used. Such a record is shown by Fig. 9 of the raw water to be Micro-

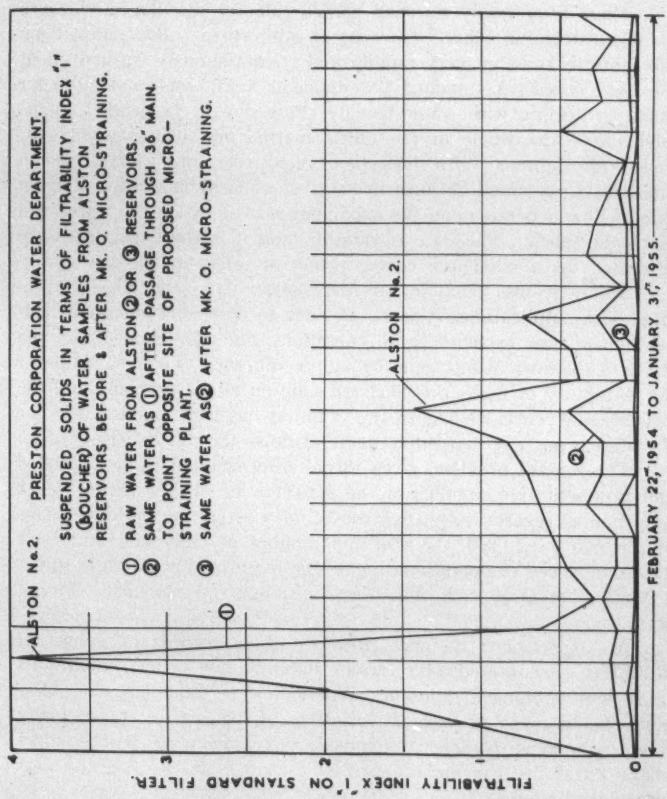

ALSTON No.2. PRESTON CORPORATION WATER DEPARTMENT.
SUSPENDED SOLIDS IN TERMS OF FILTRABILITY INDEX "1" (DOUCHER) OF WATER SAMPLES FROM ALSTON RESERVOIRS BEFORE & AFTER MK. O. MICRO-STRAINING.
① RAW WATER FROM ALSTON ② OR ③ RESERVOIRS.
② SAME WATER AS ① AFTER PASSAGE THROUGH 36" MAIN TO POINT OPPOSITE SITE OF PROPOSED MICRO-STRAINING PLANT.
③ SAME WATER AS ② AFTER MK. O. MICRO-STRAINING.
ALSTON No.2.
FILTRABILITY INDEX "1" ON STANDARD FILTER.
4
3
2
1
0
FEBRUARY 22nd 1954 TO JANUARY 31st 1955.

FIG. 9.—EXAMPLE OF FILTRABILITY RECORDS.

strained at Preston, England. This diagram also shows the effect of sedimentation in the pipeline, and the effect of Mark 0 Micro-straining.

It has been shown[9] that, for filtering media in the form of membranes, loss of head H in filtering is related to the initial head loss H_o to the volume filtered V and to the Filtrability Index I, for a given rate of flow per unit area of filter, by the expression

$$H = H_o e^{IV} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

It can be shown that six factors involved in the flow problem of an automatic self-cleaning Micro-strainer are connected by the fundamental equation (1) in such a way that:—

$$H = \frac{m Q C_f e^{nIQ/S}}{A} \quad \ldots\ldots\ldots\ldots\ldots \quad (2)$$

where H denotes the loss of head across filtering fabric, virtually the same as the differential head across the drum.

Q denotes constant rate of flow through the strainer.

C_f denotes porosity factor of fabric represented by H_o at unit velocity of approach with fabric perfectly clean.

A denotes effective submerged area of filtering fabric on drum.

S denotes speed of rotation of drum in units of effective fabric area entering the water at a given time and

I denotes Filtrability Index of raw water measured on filtering fabric in use.

The symbols n and m denote constants.

For H in inches, Q in gallons per hour (Imperial), C_f in feet, A in square feet, and S in square feet per minute, m and n have the following values:—

$$m = 0.00053 \qquad n = 0.00268$$

Fig. 10 shows a typical set of performance curves for a particular size of Micro-strainer, giving solutions to equations (2) for a range of filtering media.

Results are usually expressed at 50° Fahrenheit. Conversions to

other temperatures can be made from the principles of viscous flow, or an empirical formula, derived from Hazen's expression for flow through sand beds may be used:

$$H_t = H_{50} \frac{(\ 60\ )}{(\ T + 10\ )} \quad \ldots\ldots\ldots\ldots \quad (3)$$

Where H_t denotes the loss of head at temperature T degrees Fahrenheit, H_{50} the loss of head at 50° Fahrenheit, and T the temperature of which the loss of head is H_t degrees Fahrenheit.

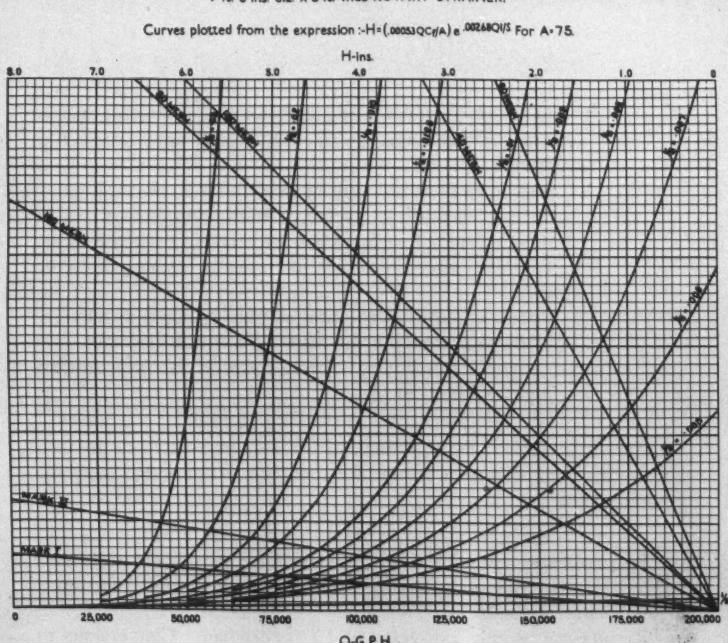


FIG. 10.—TYPICAL PERFORMANCE CHART FOR MICRO-STRAINERS.

The loss of head H in the expression (2) should be below the head at which "break-through" of the mat occurs, if maximum filtration efficiency is to be achieved. This head can be determined from tests made on a Filtrability Apparatus in laboratory conditions. "Break-through" is shown when readings of Log H plotted against V give a straight line which suddenly falls off towards the V axis.

The optimum working value of H in most Micro-straining plants is of the order of 6 inches and, as has been pointed out above, even at this low head very high flow ratings can be used without the need for pumping, even when plant has to be installed virtually on a uniform hydraulic gradient. The largest installations can be designed for an overall head loss, including channels, sluices, weirs and Microstrainers, of from 12 to 18 inches.

The Filtrability Apparatus referred to, has been described elsewhere[9] but there also exists an instrument known as the "Filtrameter", illustrated by Fig. 11, which is convenient for carrying out filtrability tests in the field.

Estimation of Filtration Efficiency

The efficiency to be expected from Micro-straining plant may be estimated from typical samples of water by means of laboratory tests made by the aid of the Filtrability apparatus. Direct filtration tests can be made by pouring the water through specimens of the various micro-fabrics, taking precautions to simulate, as far as possible, the conditions of the actual plant. This can be simply done.

A 1½" or 2" diameter tube, 6" long, open at one end, the other end covered by the fabric, provides a suitable means of obtaining filtered samples. With the tube held at an angle, the raw water, after sufficient agitation to distribute the suspended matter in its natural state and concentration, is decanted gently into the cylinder until the head rises to 5 or 6 inches or to other working head already determined, pouring finally at a rate as close as possible to the straining rate. The filtrate is collected in a clean vessel in sufficient volume for subsequent examination.

Measurements of filtrability are then made on the raw water and on the strained water, using a "Standard" filter (Institution of Water Engineers, 1948,[10, 11]). The percentage reduction of Filtrability In-

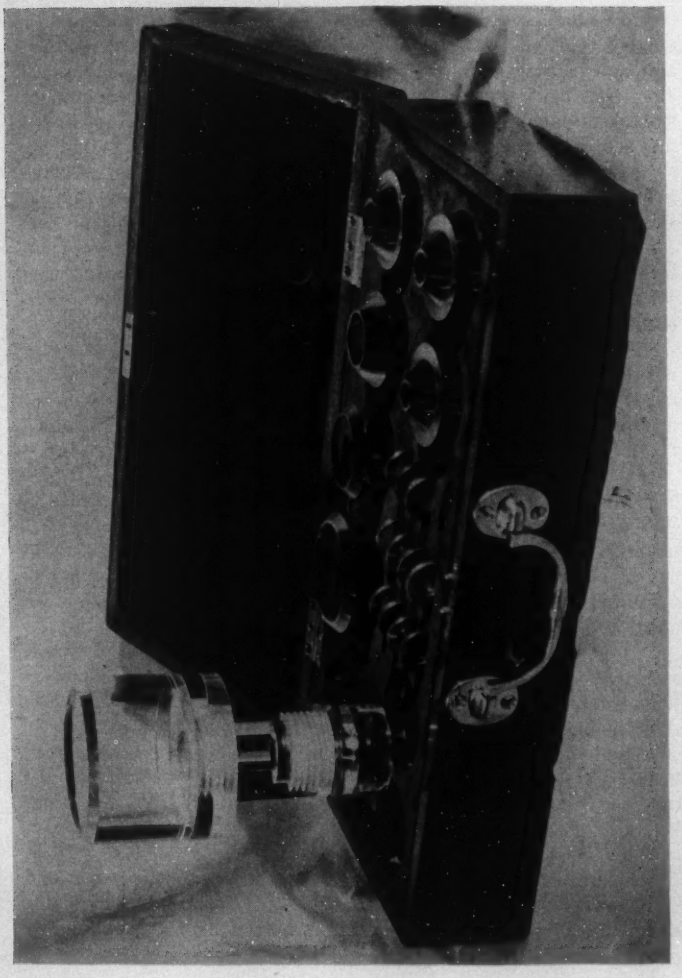


FIG. 11.—"FILTRAMETER"—FILTRABILITY-MEASURING INSTRUMENT FOR FIELD USE.

dex *I* is then a measure of the suspended solids removed by filtration, expressed in terms of the rate of blockage of the "Standard" filter. This has been done in the tests shown by Fig. 9. Estimations of dried weights of solids in p.p.m. can also be made on the raw and strained samples.

Another estimation that is often of great use is the percentage increase of Filtrability *F* in seconds, this value being obtained by calculating the reciprocal of the Filtrability Index *I*. The significance of this percentage increase is in the increased length of "run" of the "Standard" filter before and after filtration, which is taken as the improvement effected by the Micro-straining process. This method of expressing the results is of particular interest in cases where Micro-straining is to be used for primary filtration before subsequent secondary filtration, because of considerable evidence which connects improvement in filtrability seconds so measured with improvement of runs of secondary filters taking the primary filtered water. An interesting Paper dealing with this angle was published recently.[12] An exact relationship between such filtrability tests applied to predictions of secondary filter runs cannot, however, be expected. It must be remembered that biological and physical change may take place in such filters, and that additional phenomena, such as after-growths, tend to complicate matters but, nevertheless, evidence shows the value of such laboratory estimations.

For full information on the method of carrying out filtrability tests, on "Standard" filters and on associated matters, reference should be made to the published Papers and Reports (See bibliography.)

Acknowledgments

Grateful acknowledgments are made to the Engineers and Authorities, operating Micro-straining Plant, for permission to include in this Paper the information and illustrations presented.

Similar acknowledgments are made to Glenfield & Kennedy Limited, the manufacturers of the plant.

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THE ANALYSIS OF COMPLEX WATERWORKS DISTRIBUTION NETWORKS

BY EDWIN B. COBB*

[*Read March 17, 1955*]

The analysis of flow in waterworks distribution networks has interested many engineers. Since Hardy Cross presented his classic paper on network analysis in 1936 (1), there have been more than a score of papers published on the subject. Network analysis has been a sort of hobby for the writer since about 1941, when he first became interested in the subject.

It is the intention of this paper to present a system of network analysis which should prove to be a new and useful tool for waterworks engineers. Every endeavor has been made to present the material in a simple straightforward manner and to avoid repetition of what has been written before.

There are two types of waterworks networks, and to differentiate between them, the writer has coined the terms "Simple Network" and "Complex Network." A simple network is one which is supplied at a single point. A complex network is one which is supplied at two or more points by sources which have rates of input dependent upon the resistance to flow in the system.

A simple network is illustrated in Fig. 1. The source of supply of the network is indicated as a standpipe, but a pump or other source could have been substituted. Such a network can be readily analyzed by a mathematical method similar to that of Hardy Cross.

One type of complex network is illustrated in Fig. 2. The pipe arrangement is similar to that shown for the simple network in Fig. 1, but in this case the supply is from two standpipes, A and B, which are at different elevations. Centrifugal pumps could be substituted for one or both of the standpipes. For simplicity, no outflow has been considered from any of the junctions.

*Partner, Metcalf & Eddy, Engineers, Boston, Mass.

In the complex network shown, the sum of the inflows from standpipes A and B must equal the outflow from the system at C. Also, the algebraic sum of the losses in each pipe by any route from A to C must equal the available head between A and C. Similarly, the algebraic sum of the losses by any route from B to C must equal

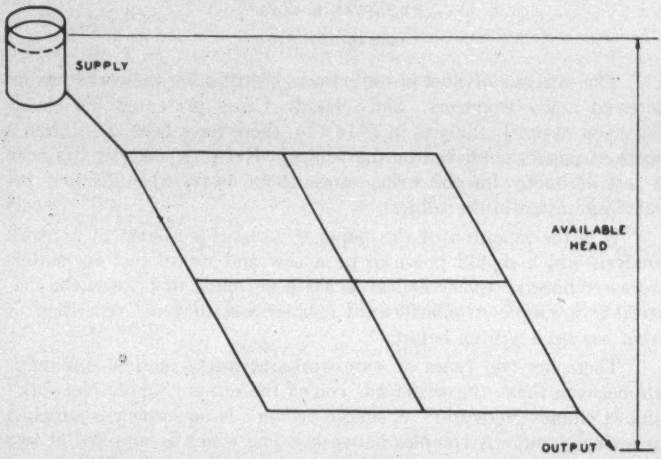


FIG. 1 SIMPLE NETWORK

the available head between B and C, and the algebraic sum of the losses by any route between standpipes A and B must equal the difference in water levels between the two tanks. It is also a fundamental rule in network analysis that the input into any junction must equal the output.

Only one combination of flows will satisfy the hydraulic conditions of the system indicated in Fig. 2, but even with such a simple network, it would be a very tedious task by trial and error to estimate that combination.

Hardy Cross (1) proposed a mathematical method for analyz-

ing networks of the complex type, but experience has shown the method to be very tedious and probably impractical for general use. The McIlroy (2) electronic network analyzer should be able to solve such a problem readily, but unfortunately many of us could not afford such apparatus, nor would we have the electrical background to be able to use it with confidence.

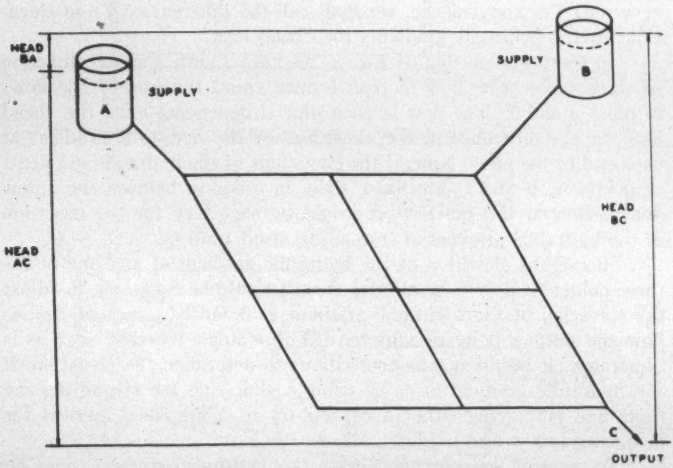


FIG. 2 COMPLEX NETWORK

As a result of considerable experimentation, the writer has developed a method for solving complex networks, which does not require expensive equipment and which is sufficiently rapid to lend itself to practical use. However, before launching into a description of the method, we should review a few fundamentals of hydraulics, to be sure all of us understand the basic principles involved.

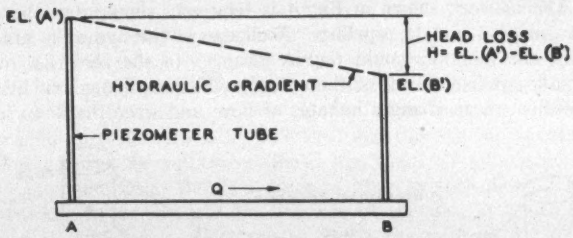
The upper portion of Fig. 3 is a simplified illustration of the hydraulic gradient for flow in a section of horizontal pipe of uniform diameter and roughness. The term (K) is a constant for the pipe, expressing diameter, roughness and length. The flow (Q)

through the pipe is induced by the difference (H) in hydraulic gradients between the two points A and B and may be computed by the expression $Q = \sqrt{\frac{H}{K}}$. By varying the difference in the hydraulic gradients, the rate of flow through the pipe may be altered or even reversed. For convenience, we shall call the difference (H) in elevations of the hydraulic gradients the "head loss."

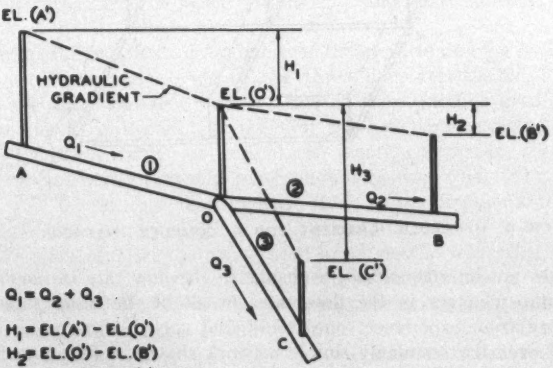
In the lower portion of Fig. 3, we have shown a simple junction of three pipes. The flow in pipe 1 must equal the sum of the flows in pipes 2 and 3. The flow in each pipe is dependent upon the "head loss" or the difference in the elevations of the hydraulic gradient at each end of the pipe. Now, if the elevations of the hydraulic gradients at points A, B and C are fixed, then, in order to balance the inflow and outflow at the junction, it would be necessary for the elevation of the hydraulic gradient at 0 to adjust itself until $Q_1 = Q_2 + Q_3$.

Should the elevation of the hydraulic gradient at any one of the three points A, B or C be altered, then it would be necessary to adjust the elevation of the hydraulic gradient at 0 until balance of the inflow and outflow is again achieved. With a single junction, such as is illustrated, it would not be too tedious to determine the elevation of the hydraulic gradient at 0 by using a slide-rule for computing the flows and employing either a cut-and-try or a graphical method for balancing inflow and outflow.

To analyze complex networks, the hydraulic gradient may be used in determining the inflow and outflow at the various junctions. The complex network of Fig. 2 is shown in Fig. 4 with the hydraulic grade lines sketched-in. If this were a problem to be solved, the elevations of the hydraulic gradients at points A, B and C would be known. Could we determine the elevations of the hydraulic gradients at the intermediate junctions, then we could compute the rates of flow in the various lines, the inflows at points A and B, and the outflow at point C. Theoretically, we could assign elevations to the intermediate junctions by trial, use a slide-rule to compute the flows, and then make adjustments in the elevations of the hydraulic gradient until the inflows and outflows of all the junctions throughout the network were in balance.



$Q = \sqrt{\frac{H}{K}}$ K = RESISTANCE FACTOR OF PIPE



$Q_1 = Q_2 + Q_3$

H_1 = EL. (A') - EL. (O')

H_2 = EL. (O') - EL. (B')

H_3 = EL. (O') - EL. (C')

$Q_1 = \sqrt{\frac{H_1}{K_1}}$ $Q_2 = \sqrt{\frac{H_2}{K_2}}$ $Q_3 = \sqrt{\frac{H_3}{K_3}}$

FIG. 3 TYPICAL JOINT

The network shown in Fig. 4 is relatively simple, but there are eight junctions and 15 pipelines. A change in the hydraulic gradient at any one junction would require changes in the elevation of the hydraulic gradient at all of the other junctions. Changes in hydraulic gradient mean changes in rates of flow, and since the flows in the

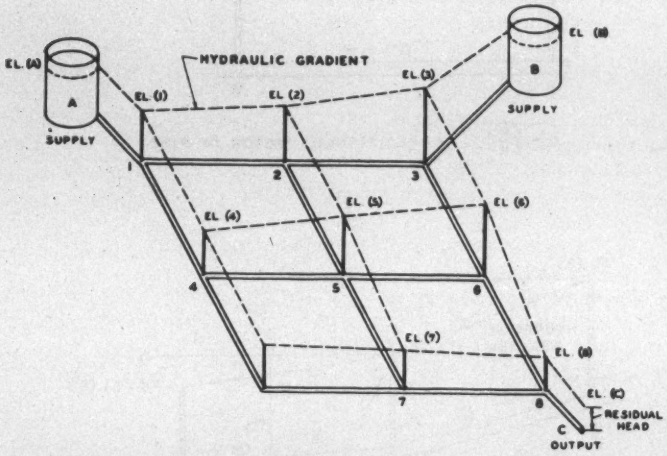


FIG. 4 HYDRAULIC GRADIENT FOR A COMPLEX NETWORK

pipelines are interdependent, a change in the flow rate in one would necessitate changes in the flow rates in all of the other pipelines. Speaking from experience, one would find such a cut-and-try solution of even the seemingly simple network shown in Fig. 4 to be no mean task.

To provide a rapid method for determining the unbalance of inflow and outflow at a junction, the writer developed a simple device consisting of paper scales and wooden disks, which, for want of a better name, he has called a "Disk Assembly." Before showing a picture of the apparatus, we shall discuss the principle upon which it works.

A group of five paper scales is shown in Fig. 5. Scales numbered 1, 2 and 3 represent like-numbered pipelines intersecting at a junction. Scale 4 is called the "index scale" and Scale 5, the "reference scale." Scales 1, 2 and 3 are laid off with lines which read in feet of "head loss" for the respective pipeline. The lines are spaced at diminishing distances from the zero line, and this spacing is equal to the flow causing the respective "head loss" laid off to a scale such as 0.5 mgd per inch. Because different pipes would normally have different resistance factors, the spacings of the feet of "head loss" lines on the respective scales would vary. The portion of the scale above the zero line represents inflow into the junction and the portion below, the outflow from the junction.

Scale 4 has a single index line with double arrow heads. Scale 5 is laid off in million gallons per day (mgd), plotted to the same linear scale as were Scales 1, 2 and 3. Scales 1 and 5 are fixed with their zero lines opposite each other. Scales 2, 3 and 4 are movable vertically.

In operation, we shall assume that the elevation of the hydraulic gradients at the outer ends of the three lines represented by the scales are fixed, but that the elevation of the hydraulic gradient at the junction is unknown. As the first step, we shall assign an elevation to the hydraulic gradient at the junction such that the head losses in the three lines would be as follows: $H_1 = +51$ ft, $H_2 = -12$ ft, $H_3 = -0.3$ ft. Plus signs indicate inflow to the junction and negative signs mean outflow.

We shall next set the above head losses on the respective scales. Scale 2 is moved upward until the arrow at its zero line is opposite +51 on Scale 1; Scale 3 is moved so that the arrow on its zero line is opposite —12 on Scale 2, and the Index Scale is moved so that its double-pointed arrow is opposite —0.3 on Scale 3. When these settings have been made, it will be seen that the double-pointed arrow on the Index Scale is opposite about —0.20 on the Reference Scale. This would mean that there is about 0.20 mgd more outflow than inflow into the junction.

To correct the unbalance between flows, the elevation of the hydraulic gradient at the junction would be lowered slightly. This would increase the inflow and decrease the outflow. By determining the

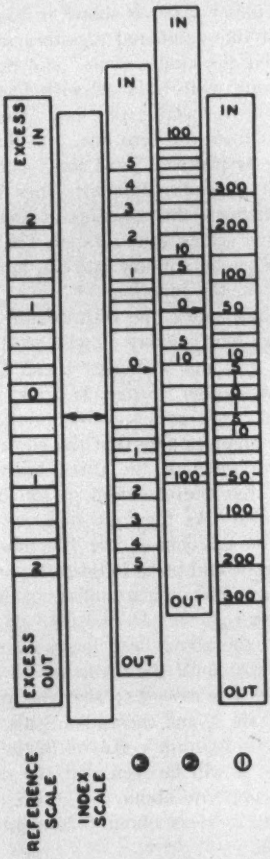


FIG. 5 SCALE ASSEMBLY

revised head losses and resetting the scales, the correctness of the revised estimate could be determined. Through successive trials, the correct elevation could be estimated.

For network analysis, the above procedure has been expanded by

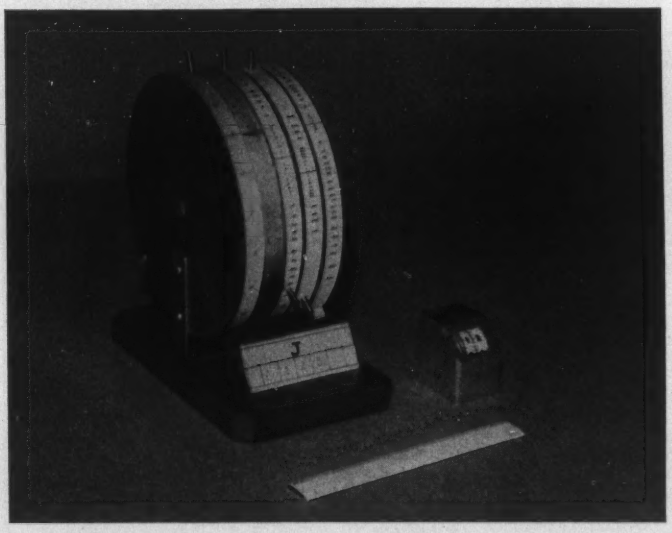


FIG. 6.—DISK ASSEMBLY AND ELEVATION INDICATOR.

mounting the paper scales on the wooden disks of a "Disk Assembly" and providing one assembly for each junction of the network. A typical Disk Assembly is shown in Figures 6 and 7.

Each assembly consists of four or more wooden disks, each about 7 in. in diameter by about ½ in. thick, mounted on a common shaft, which is supported from a wooden base. The paper scales are fastened to the peripheries of the wooden disks by means of wooden wedges, which are forced into slots in the rims of the disks. The end disks are fastened so that they remain stationary, but the intermediate

disks are free to revolve. Sheets of fine sandpaper are fastened to the right-hand faces of disks 2, 3 and 4, to provide sufficient friction between disks to cause group movement when desired. A thumbscrew and spring at one end of the shaft provide a means for adjusting the

FIG. 7.—DISK ASSEMBLY AND ELEVATION INDICATOR EXPLODED TO SHOW CONSTRUCTION.

compression on the disk assembly. The pins seen protruding from the intermediate disks are handles for rotating these disks.

The purpose of the sandpaper and group movement is to permit the adjustment of the setting on one scale without changing the settings on the other scales, except on the Reference Scale. When Disk 2 is rotated to adjust the setting on Scale 1, the sandpaper provides sufficient friction between Disks 2, 3 and 4 to cause the three to move as a group. Similarly, if the pin on Disk 2 is held with one hand to prevent the rotation of that disk, while at the same time the other

hand rotates Disk 3 to adjust the setting on Scale 2, the adjustment may be made without changing the settings on Scales 1 and 3.

When a network is to be analyzed, a simplified plan of the system would be prepared as a work sheet. A Disk Assembly would be provided for each junction to be considered. If we refer to the network of Fig. 4 as an example, we see in this case that eight junctions are to be considered; thus, eight Disk Assemblies would be required. All junctions have three pipes, with the exception of junction 5, which has four pipes, and would require an assembly with six disks.

The analysis would proceed in accordance with the following steps:

1. The length, diameter and roughness factor for each pipeline in the network would be established, and the paper scales prepared and mounted on Disk Assemblies. Since most of the pipelines are involved in two junctions, two scales would be needed for most of the lines.
2. The elevations of the hydraulic gradients at points A, B and C would be established for the problem. If A and B are standpipes, the hydraulic gradient would normally be at the same elevation as the water surfaces. Point C is the point of discharge, and if we were attempting to determine what hydrant discharge would be available, we might assume a residual pressure of 20 psi (46 ft), and from this the elevation of the hydraulic gradient at C would be computed.
3. Elevations of the hydraulic gradients at all intermediate junctions would be assigned by inspection.
4. The head loss in each pipeline would be determined from the differences in the elevations of the hydraulic gradients at the junctions, and these values would be set on the proper scales of the Disk Assemblies. The unbalance of the inflows and outflows at each junction, as indicated on the Reference Scales, would be noted.
5. Using the indicated unbalance of flows at the various junctions as a guide, we would adjust the elevations of the hydraulic gradients at the junctions. From the adjusted values, new head losses for the pipelines would be determined, the settings on the Disk Assemblies adjusted, and the revised

degree of unbalance at the junctions, as indicated on the Reference Scales, again noted.

6. The procedure of adjusting the elevations of the hydraulic gradients and making comparable adjustments in settings on the Disk Assemblies would be repeated, until the inflows and outflows at all junctions are in a reasonable state of balance. After a little practice one acquires a sort of feeling for the particular network, and the system can be adjusted to balance quite quickly.

After a state of balance has been achieved, it is prudent to check the results by computing the flow in each line from the indicated head losses. These flows may be computed from tables, by slide-rule, or by other methods, depending upon the preference of the operator. The proof of any analysis is the degree of balance of the inflows and outflows of the various junctions.

To assist in the adjusting operation, dial-type indicators called "Elevation Indicators" have been constructed. These devices are shown in Figures 6 and 7. One indicator would be provided at each junction and the elevations of the hydraulic gradients set on the dials. By mental subtraction of the dial readings of the indicators at the junctions at the ends of a pipeline, one may readily determine the head loss for the pipeline. When an elevation of the hydraulic gradient is to be adjusted, the adjustment would be made by changing the dial setting of the appropriate Elevation Indicator.

Takeoffs at various junctions are no problem in this system of analysis. Either allowance can be made for them when reading the Reference Scale, or that scale can be moved upward on the disk so that the total inflow must equal the outflow, including takeoff, before the double-headed arrow on the Index Scale will point to zero on the Reference Scale.

Probably the most time-consuming element of the analysis is the preparation of the paper scales. A simple method for making the scales is illustrated in Fig. 8. The rate of flow in a pipe is computed over a sufficient range of head losses to permit the construction of a smooth curve. Any pipe flow formula may be used which suits the operator's fancy. This curve is plotted graphically to suitable scales and mounted on a drawing board as indicated. A strip of paper, wide

enough for two paper scales and of sufficient length for the required inflow-outflow portions of the scale, is attached to the drawing board, as shown, with its zero line opposite the datum of the graph.

Ordinates are drawn to the curve for the various head losses of 1, 2, 3, ft, etc., and then, by the aid of a T-square, their intersections

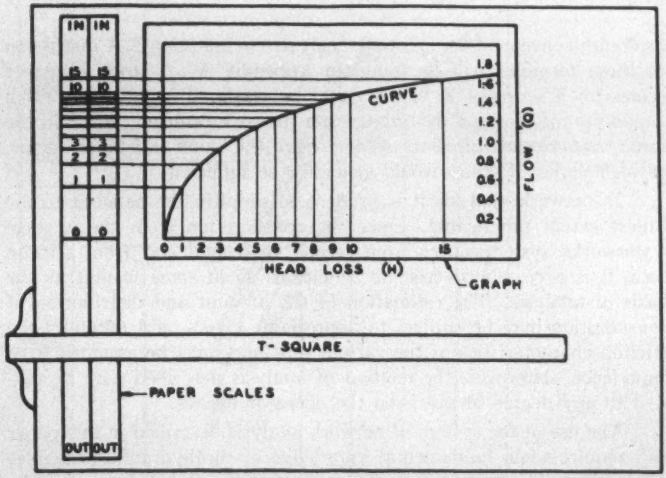


FIG. 8 BASIC METHOD OF MAKING PAPER SCALES

with the curve are projected horizontally to the paper scales and suitably inscribed. To make the lower half of the scales, the paper strips are simply inverted.

The method of preparing a paper scale for a centrifugal pump is illustrated in Fig. 9. The pump characteristic curve is transposed into terms of elevation of the hydraulic gradient at the pump discharge. By deducting from the elevations of the hydraulic gradients of the pump characteristic curve, the head loss for corresponding

rates of flow in the discharge main between the pump and the first junction, a curve representing the pump discharge at various elevations of the hydraulic gradient at the junction is obtained. This curve is then used to prepare the paper scale, as indicated in Fig. 9.

The simple formulas for pipe flow, $H = KQ^2$ *and* $Q = \sqrt{\frac{H}{K}}$, are quite convenient for network-analysis computations. A discussion of these formulas will be found in Appendix A. A stock of paper scales for a series of K values could be prepared in advance, which would permit the rapid setting-up of a network problem. Since in the same network two pipelines seldom have the same resistance factor, a single series of scales would generally be suffiicent.

In network analysis it is prudent to simplify the network to the fullest extent practicable. Since the consumption from the average waterworks system varies from second to second and from area to area, it is necessary to assume conditions as of some instant as the basis of analysis. The estimation of the amount and distribution of consumption may be subject to appreciable errors, and normally the friction characteristics of the various pipelines must be assumed from experience. However, the method of analysis described may be carried to any degree of precision the operator desires.

The use of the system of network analysis described in this paper will require some fundamental knowledge of the hydraulic principles of the flow of water in pipe. However, once the basis of the analysis is established and the paper scales are prepared, almost any person of normal intelligence and some degree of patience can be quickly taught to manipulate the Disk Assemblies, to determine the flow pattern which will satisfy the network conditions.

The writer believes that a good name for the system of analysis described would be "The Hydraulic Gradient Method of Network Analysis."

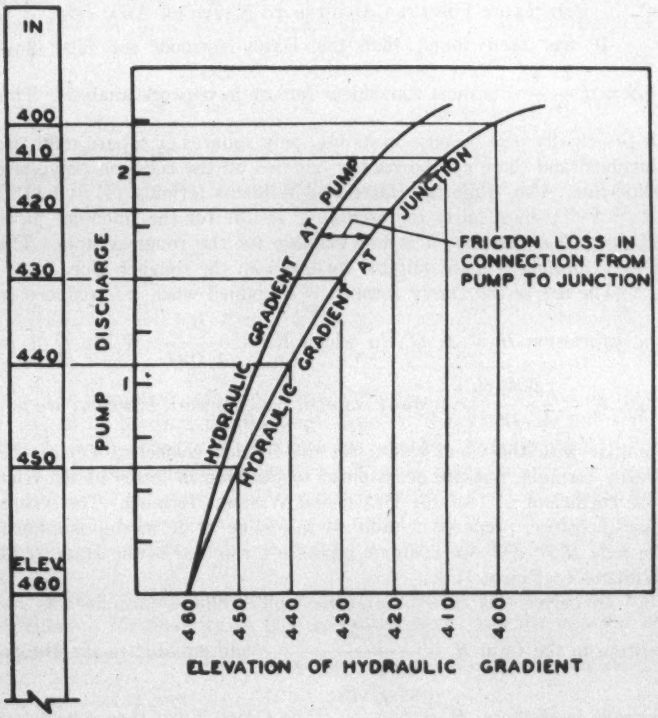


FIG. 9 METHOD OF MAKING SCALE FOR CENTRIFUGAL PUMP SUPPLY

APPENDIX A.

Pipe Flow Formula Adapted to Network Analysis

It has been found that the Darcy formula for pipe flow, $(H = f \frac{L}{D} \frac{V^2}{2g})$ is most convenient for use in network analysis. This is principally true because in its use, only squares or square roots are involved and these can be readily handled on the common polyphase slide-rule. Also, while the Hazen and Williams formula ($V = 1.318C\ R^{.63}\ S^{.54}$) gives fairly representative results for the smoother pipes ($C = 120$ or better), it is less suitable for the rougher pipes. The Darcy formula is more suitable for use with the rougher pipe.

The use of the Darcy formula is simplified when it is reduced to the expression $H = K\ Q^2$, in which $K = \frac{fL}{2gDA^2}$. When Q is in mgd, $K = \frac{.606\ fL}{D^5}$. Many waterworks engineers, however, are not familiar with the usual values for the friction coefficient (f) of the Darcy formula, but are accustomed to thinking in terms of the friction coefficient (C) of the Hazen and Williams formula. The writer has, therefore, prepared a table of the value of K in the simplified formula $H = KQ^2$ for common pipe sizes, referred to the Hazen and Williams coefficient (C).

To derive this table, the Hazen and Williams formula was rewritten in the form $H = \frac{LV^{1.85}}{1.667\ C^{1.85}\ R^{1.16}}$ and equated to the Darcy formula in the form $H = f \frac{L}{D} \frac{V^2}{2g}$. The Darcy f was then solved for in terms of the Hazen and Williams C as follows:

$$f \frac{L}{D} \frac{V^2}{64.4} = \frac{LV^{1.85}}{1.667\ C^{1.85}\ R^{1.16}}$$

$$f = \frac{193.2}{C^{1.85}\ D^{0.16}\ V^{0.15}}$$

Values of f were computed for 8-in., 12-in. and 18-in. diameter pipe at velocities of 3, 4 and 5 ft for a series of values of C ranging from $C = 40$ to $C = 150$. Some of these values are tabulated in Table A-1.

TABLE A-1. VALUES OF DARCY f

Diameter of pipe (in.)	Velocity (fps.)	Value of H. & W. C	Value of Darcy f		
			150	100	40
8	5		0.0152	0.0325	0.176
8	4		0.0157	0.0336	0.183
8	3		0.0163	0.0349	0.189
12	5		0.0142	0.0304	0.165
12	4		0.0146	0.0314	0.171
12	3		0.0153	0.0328	0.178
18	5		0.0133	0.0286	0.155
18	4		0.0139	0.0297	0.161
18	3		0.0145	0.0310	0.169

The computed values of f have been plotted in Fig. A-1 and are found to lie between the limiting curves of the 8-in. pipe and 3 fps velocity and of the 18-in. pipe at 5 fps velocity. Average values for f at various intervals of the value of C were taken from the curves, and these values were used to prepare Table A-2. In this table are tabulated K values for the Darcy formula per 1,000 ft of pipe in sizes from 4 in. to 60 in., inclusive, and with C values from 40 to 140, inclusive.

In the simplified form, the Darcy formula lends itself to easy calculation for quantity of flow or loss of head, and the solution of pipe in series or in parallel.

1. *Single pipe*
   a. Loss of head $H = KQ^2$
   b. Quantity of flow $Q = \sqrt{\frac{H}{K}}$
2. *Equivalent friction coefficients K*
   a. Pipe in series $K_T = K_1 + K_2 + K_3 + \dots K_n$
   b. Parallel pipe $K_e = \dfrac{1}{\left(\dfrac{1}{\sqrt{K_A}} + \dfrac{1}{\sqrt{K_B}}\right)^2}$

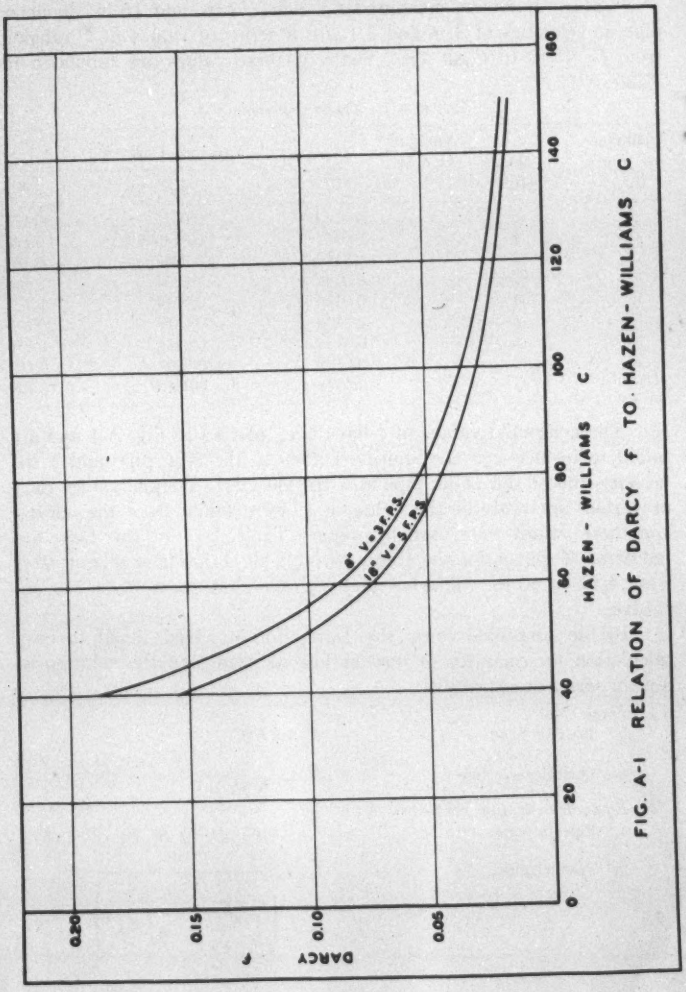


FIG. A-1 RELATION OF DARCY f TO HAZEN-WILLIAMS C

TABLE A-2. VALUES OF DARCY K

Pipe size (in.)	H&W "C" / Aver. value Darcy "f"	Values of Darcy K* per 1,000 ft of pipe					
		40	60	80	100	120	140
		0.170	0.080	0.048	0.030	0.022	0.016
4		2520.	1180.	710.	444.	326.	237.
6		330.	155.	93.1	58.2	42.7	31.0
8		78.0	36.7	22.0	13.8	10.1	7.34
10		25.7	12.1	7.25	4.53	3.32	2.42
12		10.3	4.85	2.91	1.82	1.33	0.970
14		4.78	2.25	1.35	0.843	0.618	0.450
16		2.45	1.15	0.691	0.432	0.317	0.230
18		1.36	0.638	0.383	0.239	0.176	0.128
20		0.799	0.376	0.226	0.141	0.103	0.0752
24		0.321	0.151	0.0907	0.0567	0.0416	0.0302
30		0.105	0.0496	0.0298	0.0186	0.0136	0.00992
36		0.0423	0.0199	0.0120	0.00747	0.00548	0.00398
42		0.0195	0.00920	0.00552	0.00345	0.00253	0.00184
48		0.0101	0.00474	0.00284	0.00178	0.00130	0.000947
60		0.00330	0.00155	0.000931	0.000582	0.000427	0.000310

*Q in mgd.

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SUMMER WATER AT ROCKPORT

BY MATT HAUTALA*

[*Read June 16, 1955.*]

This paper deals with some of the problems of a water department in a summer-resort town, where the demand for water tends to exceed the supply. Rockport, Mass., is such a town and is located on the tip of Cape Ann, having for its only neighbor the city of Gloucester.

It has a winter population of 4,200 and an estimated summer population of nearly 15,000. Back in 1893, when the Rockport Water Department first came into existence, the town acquired a large surface pond, called Cape Pond, as the first source of supply. This pond covers an area of some 50 acres, has a watershed of over 200 acres, and is 21 ft in depth at the deepest point. It is believed to be fed by springs. At one end of the pond is a swamp, which undoubtedly contributes to a water high in color and noted for its algae growth and swampy taste. The estimated safe yield of this pond is some 150 mil gal per year. In addition to the pond, the water works originally consisted of a steam pumping station, a 317,000-gal standpipe and approximately 8 miles of water mains, with 175 paying customers. Through the years the system has continually expanded, until today we have over 22 miles of mains, 6 miles of summer pipes and 2,300 water services. Thus it can be seen that the town has continued improving and increasing its water system.

Because of the poor quality of the Cape Pond supply, it was recommended to the town that improvements be made by constructing a filtration plant, to correct a bad situation which had existed for a number of years. In 1938 such a project was undertaken with Federal funds and a 1-mgd filtration plant was designed and constructed. This station has two rapid-sand filters, each of ½-mgd capacity, and the entire station is electrically operated. The raw water is pumped

*Superintendent of Water and Sewers, Rockport, Mass.

from the lower-lift station, which was the original old steam plant and is now fully electrified, into the filtration plant, where the necessary chemicals are added. At the present time it is found that pre-chlorination, pre-lime, alum, activated carbon, post-lime and post-chlorination work most advantageously for the filtration process. Especially during parts of the summer months as much as 16 ppm of chlorine is necessary for pre-chlorination, in order to cope with the algae, even after several doses of copper sulphate, equivalent to 6 lb per mil gal, are applied to the pond. It is believed that the raw water is perhaps the most difficult to process in this section of the country, and full credit is given to the operators, who from experience are furnishing a finished water which is low in color, of excellent quality and far superior to what the town had during its earlier days of operation.

As another source of supply the town made studies and undertook steps to develop a driven-well field, and now has 21 tubular wells, averaging 26 ft in depth. From this source some 55 mil gal are obtained each year. This auxiliary source was intended to furnish the town's needs during the winter and spring, and thereby minimize the amount needed from Cape Pond. For the most part this nearly meets the requirements of the town. In recent years, however, there has been a shortage of water, because of increased demand, due to extensive sewer installations, renewal of a large number of old water services, and influx of people who have settled permanently in Rockport. Additional demand comes from those who have established summer residences in the town, as well as from the week-end tourist trade.

In 1946, when the writer became Superintendent, he foresaw the need for more water during the summer months. At times on very hot days, our actual demand exceeded the total pumping output of both the filtration plant and the auxiliary well fields. On a number of occasions the standpipes would be practically empty, and this left the town in a very critical position in case of fire or other emergency requirements. Under these circumstances, the town had to choose between two courses of action, in order to remedy a serious condition which would in a very short time face the Water Department.

1. We looked into the feasibility of installing water meters. On first thought this seemed to be a solution to our problem. Upon fur-

ther investigation it became clearly evident that water meters would perhaps serve only as temporary stop-gaps for what would later develop. Of the number of water services approximately 25% are to summer dwellings, and people who come to Rockport to spend their summer expect that they will be able to maintain decent grounds about their property. Many have nice flower gardens; others have vegetable gardens, and the majority have nice green lawns. Since it was felt that the public would not be interested in curtailment of the amount of water which they could use, it was finally decided that water meters would probably not be the answer.

2. A study was made of the possibility of further augmenting the water supply from a third source. A number of plans were investigated, and finally it was decided that the proper procedure would be to develop some of the abandoned quarries for reservoir purposes. The town of Rockport contains a number of such quarries, as a result of an extensive granite-quarrying industry during the 19th and early 20th centuries. Millions of tons of granite were quarried and shipped to remote places, leaving huge quarry holes some 200 ft in depth, with a surface area of 6 acres. As the granite industry faded out about the year 1929, these quarries became idle and filled with water. It was felt that some of these quarries could be utilized as open reservoirs and that during periods of excessive demand the water in storage could be used. Permission was granted by the State Department of Public Health to seek this additional water from the quarries. It was indicated that the supply was of excellent quality and safe in all respects. Very little opposition from the voters resulted, and a program was started whereby the town purchased three quarries having an available watershed of some 120 acres. This property was acquired at a nominal price of $15,000.

The Water Department proceeded with caution and in the first year set up a temporary pumping station with a centrifugal pump, borrowed from the auxiliary station. Water was pumped from the quarries directly into the mains. The only treatment necessary was the addition of chlorine for safety reasons, as required by the Health Department. Careful checks were kept, and during the summer of 1951 the Water Department was able to furnish the requirements of all its customers. No longer was it necessary to curtail the

use of hose and sprinkler and, in general, the public appreciated what the town was trying to accomplish.

During the fall we carefully watched the recovery rate and found that, by diverting two brooks into the quarry, we had ample recovery within a short time. The following year further steps were taken to improve this locality, and a masonry dam, some 30 ft in height and 60 ft in length, was constructed across a gap. This improvement made it possible to increase the storage facility of this particular quarry by an estimated 75 mil gal. Shortly after the dam had been completed, seasonal fall rains came, and within a few weeks the quarry had recovered all that the Department had pumped during the summer and was filled to capacity.

During the years 1953 and 1954 we used this new source of supply continuously from early April until late in the fall, and each year the recovery has been excellent. It is now planned to construct a permanent pumping station, having turbine pumps and being capable of operation in winter, in the hope that this new source of supply will eventually become the main source of water for the town.

In conclusion, it may be pointed out that what at first appeared to be almost an impossibility has gradually been developed into something which the town can consider itself fortunate to have. Our sincere hope is that the water situation in Rockport has been corrected to such an extent that the general public will be more than well satisfied that the water in Rockport will be sufficient without further restriction.

REALISTIC LIMITATIONS OF FLOW-MEASUREMENT RANGEABILITY AND TOLERANCE

BY LELAND K. SPINK* AND RICHARD C. TAMM†

[*Read September 21, 1955.*]

It is doubtful if there are any two words in the English language which are subject to as wide a variety of interpretation as the words "rangeability" and "tolerance". The application of these terms to flow meters has been a constant source of confusion to both the user and the manufacturer.

To the manufacturer, the simplest expression of flow-meter calibration uncertainty is in terms of per cent of the full scale of the meter. This value is most logically a constant figure, regardless of the point of operation on the meter scale, because the usual bugaboos of manufacturing are friction and lost motion, and these are likely to be as great at 10% of the scale as at 100%.

To the meter user, on the other hand, if he plans to use the meter most of the time at readings of 20% or 30% of the scale, the calibration tolerance at full scale is less important than the meaning of this tolerance in terms of flow at his operating conditions. There would seem to be a real need for a means of expressing the manufacturer's guarantee in terms which have a useful meaning to the user.

The use of the word "tolerance" in the sense intended by the authors is a moot point, because of the common association of this term with the machining practice, in which the term "tolerance" means the maximum allowable error. However, it will be found that the Webster definition of the term "tolerance" does not tie it to a maximum value. The term has been adopted by the International Standards organization as applied to a probable variation in the coefficient of a primary device, and seems to apply as well to the uncertainty of overall measurement results.

The Power Test Codes Flow Material Subcommittee of the

*Engineer in Charge of Flow Measurements, The Foxboro Co., Foxborough, Mass.
†Product Engineer, The Foxboro Co., Foxborough, Mass.

ASME Fluid Meters Research Committee was given the task of assigning coefficient tolerances to different types of primary devices a number of years ago. The discussions in this Subcommittee brought out the fact that even in teaching and scientific circles there was no standard mathematical meaning for the word "tolerance". A root-mean-square value, calculated from the data, was used in some cases. The probable error, which means that as many of the results would be less than the probable error as would exceed it, was sometimes used. However, in scientific work the most common interpretation was the value which would insure that less than 5% of the results would fall beyond the specified error. The meter user is probably most interested in the maximum value, but mathematically this would approach infinity and would thus be meaningless. The 95% probability value seemed to the members of this Subcommittee to be the nearest mathematical approach to what the meter user wanted and was adopted.

For a starter, suppose we use the definition of measurement tolerance as the percentage within which 95% of the flow measurements with a given meter would fail. Without due thought given to the matter, one might say that this would be something less than the maximum error observed in the calibration of the instrument, but this is not necessarily true. Most calibrations are performed under reasonably ideal conditions in the instrument factories. Therefore, it should be realized that under field operations, where ideal conditions do not always exist, additional errors must be considered. Such things as expansion and contraction of mercury with ambient temperature changes; expansion and contraction of charts with relative humidity; flow-condition changes affecting primary-device coefficients, and vibration effects may all cause meter uncertainties.

In applying this theory, if one source of error or uncertainty is highly predominant, the effect of other small sources of variation are minimized in the over-all result. At high rates of flow, the coefficient tolerance may predominate, but at low rates of flow it is quite likely that the instrument-calibration tolerance will smother all other sources of uncertainty.

Thus, we come to the question of a definition of "rangeability". Without a clear definition of "rangeability", a manufacturer could

claim that his meter had infinite rangeability, because it would read from zero up to full scale, and the ratio of maximum reading to minimum reading would be infinity. Fortunately, we know of no manufacturer who is so unscrupulous. However, there is still a tremendous latitude for degrees of conservatism of rangeability claims. It would seem, from the user's standpoint, that he is interested in the ratio of maximum measurable flow to the minimum flow at which the meter will give readings within his allowable tolerance. Thus, one user who requires a very low tolerance will have a meter rangeability of 3:1, whereas another user, who is not as interested in accuracy at low readings, will accept a higher allowable tolerance and will obtain from the same meter a rangeability of 10:1. Therefore, to impart a meaning to the expression, "rangeability", a statement of the measurement tolerance at the minimum flow rate must be included.

An intelligent study of the subject of rangeability requires a clear distinction between multiplication errors and constant errors. A pure multiplication error is zero at zero flow and has no effect on rangeability. Errors caused by friction, lost motion, and similar effects, as well as by inability to detect small motions of the pen, may be just as large in terms of instrument scale at zero as at full scale. This does not mean that the error in terms of per cent of flow is constant; quite the contrary. The per cent error approaches infinity at the zero reading. This is the type of error or uncertainty which imposes realistic limitations on the rangeability.

This can better be understood by referring to Figure 1. In the example a gray shaded area at the top has been used to represent the possible error at full scale. The gray shaded area below represents this same error; only the pen position has changed. The upper border of the shaded areas is the true reading; the lower border is the actual reading. In the example a pen travel of five inches has been used and an assumed error of 0.010". This represents 0.2 of 1% of full scale. Three scales are shown: pen travel (0-5")—uniform scale (0-100)—and square-root scale (0-10).

The 0.010" error is slightly larger than the upper line on the chart (0.008") in Figure 2. As the constant theoretical error is observed at the lower reading, its value on the pen-travel scale remains the same—0.010". The value is also the same on the uniform scale—

UNCERTAINTIES DUE TO WIDTH OF PEN LINE

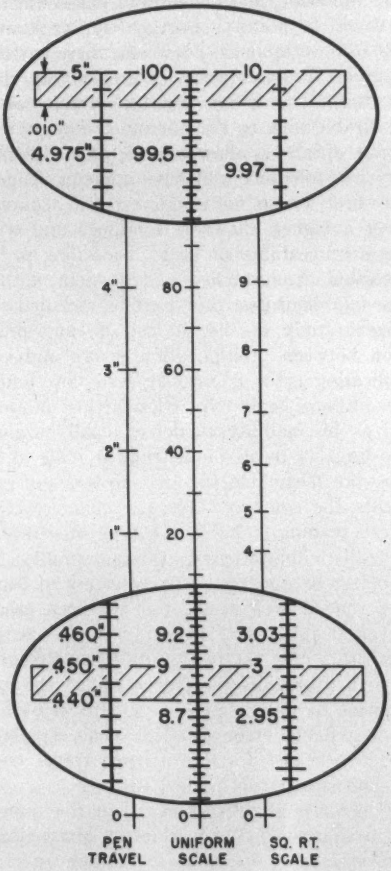


FIGURE 1.

0.2—but on the square-root scale the value is considerably more than the original 0.01. Actually it is 0.04.

Now on the uniform scale the pen should be indicating 9.0, but is actually indicating 8.8. If we divide the difference by 9.0 and mul-

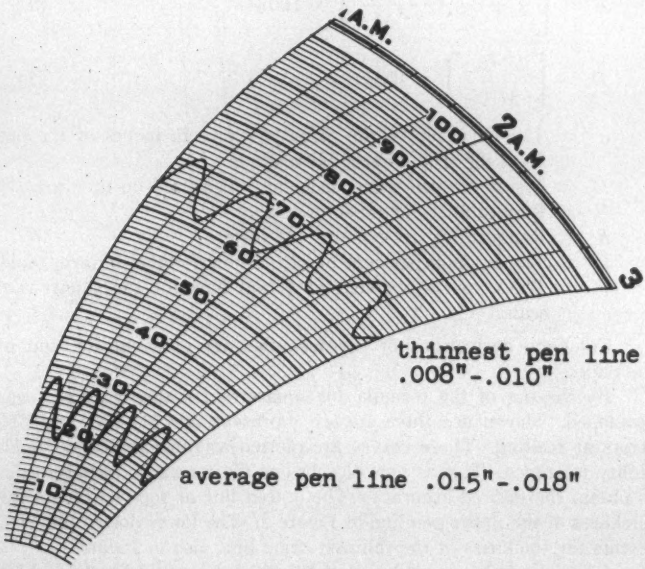


FIGURE 2.

tiply by 100, an error at reading of 2.2% is obtained. Thus, the theoretical error as a percentage of actual flow is approximately ten times as large at this low flow as at full scale. On the square-root scale the pen indicates 2.96 instead of 3.00, or a 1.33% error at reading. The rangeability of an instrument using these lower values as limits would be 11.1 for the uniform scale and 3.3 for the square-root scale.

In order that a user can determine his own rangeabilities, two

formulas are introduced. If the pen travel and error, both expressed in inches, are known, it is a simple matter to determine either the rangeability or the per cent error at the minimum reading.

$$F = \left[1 - \sqrt{1 - \frac{R^2 e}{M}} \right] \times 100 \qquad (1)$$

$$D = \left[\frac{U\ e}{M} \right] \times 100 \qquad (2)$$

e = The deviation or the actual distance in inches of the pen from the true differential reading.
M = Maximum pen travel in inches, measured on time arc.
U = Rangeability for uniform scale.
R = Rangeability for square-root scale.
D = The error in per cent of actual flow for the uniform scale.
F = The error in per cent of actual flow for the square-root scale.

Complete derivations of the formulas are given at the end of the paper.

By the use of the formula for square-root scales, Figure 3 was developed. Shown are three curves, representing 1%, 2%, and 3% errors at reading. These curves are plotted as rangeability vs. readability tolerance. That is how closely one must read a chart in order to obtain the desired accuracy. The dotted line at top represents the thickness of the upper pen line in Figure 2. The lower dotted line represents the thickness of the thinnest chart line, also in Figure 2.

As an example, a man has a meter which he would like to use for a rangeability of 10:1. He cannot have an error at reading in excess of 2%. What is his tolerance? If we check the curves, it is apparent that he must read within 0.002" of the true reading. It should be mentioned here that the M has been taken as 5", because 5" is a larger pen travel than that used by any well-known meter manufacturer. Therefore, the errors shown are actually *less* than would exist with an actual meter.

Figure 4 shows the same information for the uniform scale.

In the matter of readability, the uniform flow scale meter has

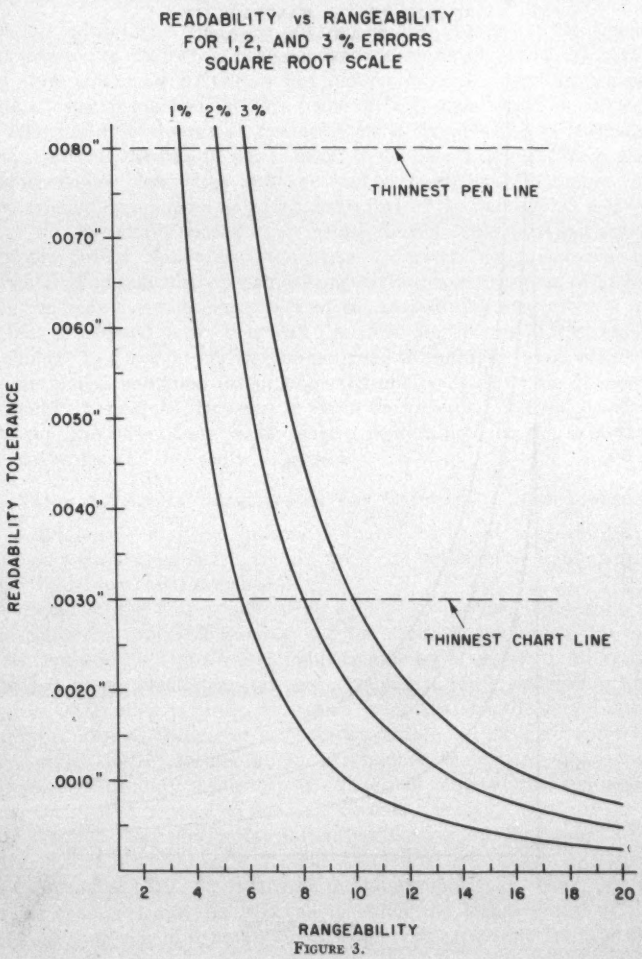

READABILITY vs RANGEABILITY
FOR 1, 2, AND 3 % ERRORS
SQUARE ROOT SCALE
1% 2% 3%
.0080"
.0070"
.0060"
.0050"
.0040"
.0030"
.0020"
.0010"
READABILITY TOLERANCE
THINNEST PEN LINE
THINNEST CHART LINE
2 4 6 8 10 12 14 16 18 20
RANGEABILITY

FIGURE 3.

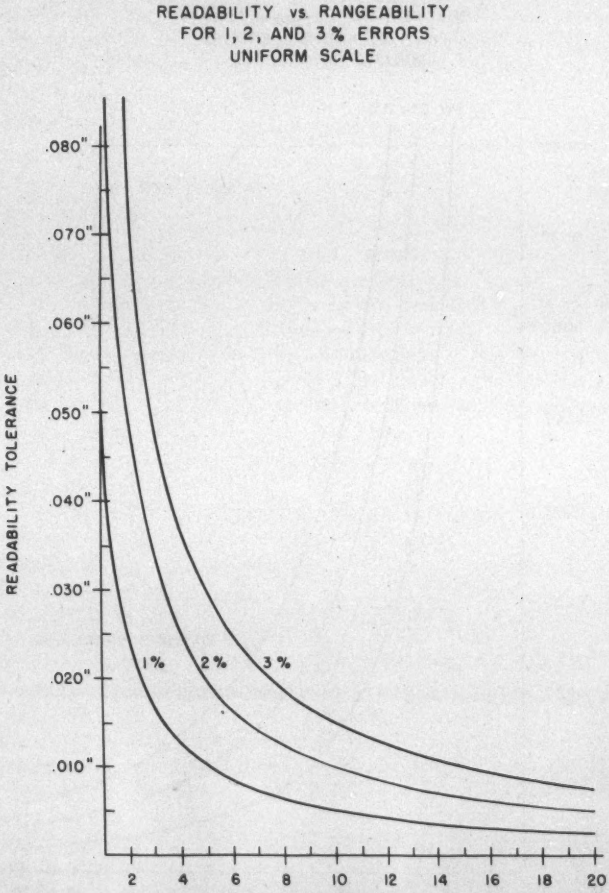

READABILITY vs. RANGEABILITY
FOR 1, 2, AND 3 % ERRORS
UNIFORM SCALE
READABILITY TOLERANCE
.080"
.070"
.060"
.050"
.040"
.030"
.020"
.010"
1%
2%
3%
2
4
6
8
10
12
14
16
18
20
RANGEABILITY

FIGURE 4.

distinct advantages, but these may be highly illusory. If the signal transmitted to the pen is in error, because of zero-error effects, temperature effects, or friction or lost motion, from an operating standpoint lower readability at this point on the scale might be better. Differential-type meters all operate from a signal which is basically the square of the rate of flow; hence, to produce a uniform scale, the signal at low flow rates must be highly multiplied. Therefore, to the original errors must be added the errors of the multiplying means.

Fundamentally linear type meters do not have this limitation. (Note: Orifice, nozzle, or venturi-type meters are not fundamentally linear.) The difficulty of maintaining tolerances in this type of meter only increases as the first power of the rangeability attempted. One might be inclined to say that with electronic multiplying means, which eliminate lost motion and can be designed to multiply the flow signal to an almost unlimited extent, the rangeability of this type of meter would be boundless. However, even in the case of electronic devices, we run into noise levels which place a definite limit on the minimum signal which can be used with accuracy.

Extending the Rangeability of the Differential-Type Meter

While this study has indicated limitations in the rangeability of the differential-type meter, certain steps can be taken to obtain a real and actual increase in rangeability.

One of the first points to be emphasized is the importance of choosing the proper flow range for the existing conditions. All too frequently in the water-works industry a meter is chosen to take care of flow rates which may not be encountered until years after the initial installation is made. It is then found that for the actual operating conditions the meter is operating barely off the zero point on the scale. Better results would be obtained if it were planned to change the primary device or the differential range at the time when the anticipated increase in flow rate became a reality.

An ingenious device for instantly changing the flow range of a meter is shown in Figure 5. The differential instrument has three mercury chambers. By means of the piping manifold valves, one of these chambers may be switched to either the high-pressure or the low-pressure side of the manometer. When the extra chamber is on the

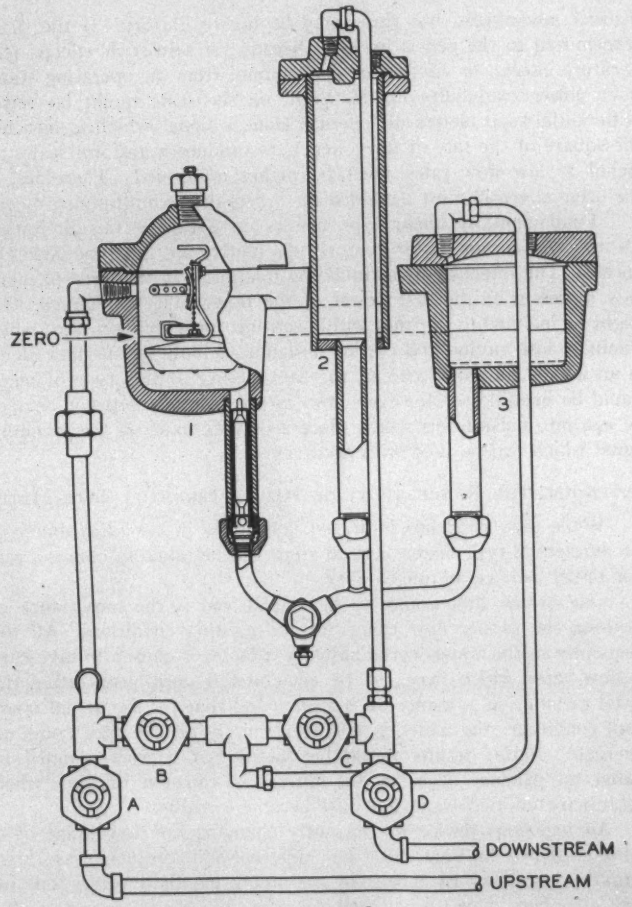


Figure 5

low-pressure side of the manometer, the instrument becomes a low differential range and, when on the high-pressure side, it becomes a high differential range, since the mercury must move farther for a given displacement of the float. The common range combination is 25" and 100". This gives a 2:1 ratio flow capacity. When a single pump is adequate for normal demands, but an additional pump must be added for high-demand conditions, the meter may be kept operating in the optimum part of the scale under both operating conditions. The design is such that no mercury is trapped in any part of the system at any time; hence, the need for mercury-tight shutoff valves and the need for zeroing the meter when changing the range are eliminated.

Mutiple meter runs may be used to obtain a real and unlimited incease in rangeability. As the rate of flow increases, additional meter runs are cut into service, to bypass part of the flow from the primary meter run, and vice-versa on decreasing flows. This operation may be performed either automatically or manually, as desired. The increase in rangeability is geometric; that is, the rangeability of two meter runs in parallel is approximately equal to the square of the rangeability of each meter. The rangeability of three meter runs is approximately equal to the cube of that of the single meter, etc.

A limited increase in rangeability may be obtained by the use of multiple recorders, operating from a single primary device. The highest range which can be used is, of course, limited by the allowable pressure loss, and the lowest range by the availability of accurate low-range differential recorders. A minor limitation is imposed by the variation of the discharge coefficient of the primary device at low Reynolds numbers. With the use of several differential instruments, operating from a single primary device, the difficulty of integration and overrange protection is increased. If integrators are used, means must be provided for switching integrator operation from one instrument to another, as the flows exceed the successive scales of the various differential ranges, and the overranged instruments must be adequately protected against the blowing of mercury or damage to the working parts.

Conclusions

Rangeability statements are meaningless unless the permissible measurement tolerance at the minimum flow rates is specified. A user who can permit a 10% uncertainty at minimum flow rate can use a given meter over a greater range of maximum to minimum flows than one who requires 2% accuracy under all operating conditions.

The laws of nature and physics impose certain limitations on the rangeability and tolerance of flow-measuring devices. It does not answer the problem to shut our eyes to the facts of the situation and demand of the manufacturer specifications which obviously cannot be met.

There are three basic approaches to increasing rangeability without increasing tolerance: increasing the sensitivity to small flow signals, use of fundamentally linear flow meters, and use of multiple meter-run installations. Under the first classification, shaped chambers and other linearizing devices increase rangeability by increasing readability and sensitivity to low flow signals. Multiple recorders, operating from the same primary device, provide the same advantages without some of the difficulties inherent in linearizing devices. The fundamentally linear flow meter gives rangeability equal to the square of that of a square-root flow-scale type having the same basic signal-measuring tolerance. The multiple meter-run installation offers unlimited rangeability possibilities.

There is one precaution which may be taken to insure obtaining the greatest accuracy and rangeability possible with a given meter, and we shall emphasize this again: choose a flow range just as near as possible to the maximum existing flow rate.

Appendix—Derivation of Formulas

The following letters are used in the formulas:

e = the deviation or the actual distance in inches of the pen from the true differential reading.

M = maximum pen position in inches from zero circle, measured on time arc.

m = minimum pen position in inches from zero circle, measured on time arc.

r = any length of pen position in inches from zero circle, measured on time arc.

U = the rangeability of a uniform scale expressing the variable number; *e.g.*, rangeability is usually expressed as a ratio, 4 to 1. For these calculations it will be expressed as U to 1.

R = the rangeability of a square-root scale.

D = the error in per cent of actual flow for the uniform scale.

F = the error in per cent of actual flow for the square-root scale.

Rangeability of a uniform scale can be expressed as:

$$U = \frac{M}{m} . \tag{3}$$

As an example, if the maximum pen travel is 4", and the minimum point at which a reading can be taken is 1", substituting in the formula we would get:

$$U = \frac{M}{m} = \frac{4}{1} = 4, \text{ or rangeability of 4 to 1.}$$

In the case of the square-root scale, the $\frac{M}{m}$ is under a radical sign:

$$R = \sqrt{\frac{M}{m}} ;$$

therefore $R^2 = U$. (4)

This presents our first major problem in rangeability when using a square-root chart. In the example above, we had a 4-to-1 rangeability on a uniform scale; on a square-root scale this would only be 2 to 1. Therefore, the range has been reduced by ½ just by introducing the square-root scale. Because of the characteristics of the square root, the effect will increase with range increases.

A formula must now be developed for determining the limiting

factor and its error. First, we shall evolve the uniform-flow-scale formula, beginning with the fundamental formula:

$$D = 1 - \frac{r \pm e}{r} .$$

It can be seen that as r increases, the effect of e becomes less. Conversely, as r decreases, the effect of e increases.

Since we are interested in D when e has the greatest effect, r will be considered at a minimum value, which is m.

Therefore, $$D = 1 - \frac{m \pm e}{m} .$$

From (3) $$m = \frac{M}{U} .$$

Then $$D = 1 - \frac{\frac{M}{U} \pm e}{\frac{M}{U}} = 1 - 1 \pm \frac{Ue}{M}$$

$$D = \left[\frac{Ue}{M} \right] \times 100 . \qquad (2)$$

To develop the same type of formula for square-root scale, our measurements must now be placed under a square-root sign, and a designation F will be used to express the error for the square-root scale.

Therefore, $$F = 1 - \sqrt{\frac{r - e}{r}}$$

Again let us substitute m for r:

$$F = 1 - \sqrt{\frac{m - e}{m}} .$$

Now substituting in formulas (3) and (4):

$$R^2 = U = \frac{M}{m} .$$

Therefore, $m = \dfrac{M}{R^2}$.

Then $$F = 1 - \sqrt{\frac{\dfrac{M}{R^2} - e}{\dfrac{M}{R^2}}}$$

$$F = \left[1 - \sqrt{1 - \frac{R^2 e}{M}} \right] \text{ x } 100. \qquad (1)$$

Formula 1 can now be used to develop the limits or rangeability and to show the error introduced by making R too large.

METROPOLITAN DISTRICT SOURCES OF WATER SUPPLY

BY WARREN A. GENTNER*

[*Read April 21, 1955.*]

October 23, 1955, will mark the one hundredth anniversary of the inauguration of municipal water supply service in Hartford.

During the first 75 years of its operation the system was owned by the city of Hartford, but on July 1, 1930, its ownership and operation were transferred to the newly formed Metropolitan District of Hartford County, a municipal corporation created by special act of the General Assembly of the State of Connecticut at its 1929 session, having as its principal functions water supply, sewerage and regional planning.

The District is presently comprised of the city of Hartford and six nearby towns. Water-supply service is also provided to two non-member towns.

The intent of this paper is to sketch briefly the growth of the system, as reflected in the development of its sources of supply, and to describe at greater length the proposed Hogback development on the West Branch of the Farmington River, construction of which will be started within the next few months.

The Connecticut River Supply

At the beginning of local water-supply operations one hundred years ago, Hartford was a city of some 20,000 to 25,000 people. The original water-supply system consisted essentially of a pumping station and intake works on the west bank of the Connecticut River, from which water was pumped through a 16-inch cast-iron force main to an open 7-mil gal storage reservoir, located about a mile west of the river and 125 ft above it. From this reservoir water was distributed by gravity through a distribution-main network to the individual consumer.

*Deputy Manager and Chief Engineer, Water Bureau, Metropolitan District, 115 Broad St., Hartford, Conn.

The Connecticut River works were in active use until 1867, when they were decommissioned but retained for standby service, though not used, until they were demolished in the early 1920's.

The West Hartford or Talcott Mountain Supply

Some dozen years after the Connecticut River works were put into operation, the city, faced with the necessity of increasing its water supply, wisely decided to abandon further use of Connecticut River water and to develop a new gravity supply in the town of West Hartford, on the eastern slopes of Talcott Mountain, some six miles west of the city.

Six small reservoirs were ultimately built in this area between 1867 and 1895. Of these six reservoirs, two are no longer used for water-supply purposes and the remaining four, with a capacity of about 1¼ billion gallons, are operated principally as emergency reserve supplies, with just enough regular use to avoid waste by spillage.

The Farmington River Watershed Supplies

By 1909 the Hartford system was supplying a population of 95,000. Water use averaged 7 mgd and the need for additional sources of supply was becoming increasingly more urgent.

Following thorough investigation of several watersheds both east and west of the Connecticut River, the Farmington River valley was finally chosen as offering the greatest possibilities for future water-supply development.

At about this time Caleb Mills Saville began his long association with this system as its Chief Engineer, and it was Mr. Saville who envisioned the entire Farmington River water-supply development program and carried to completion two of the major projects comprising that program. The Metropolitan District is indeed fortunate in having had the benefit of this eminent engineer's services.

The District's two principal water-supply reservoirs have since been developed on tributaries of the Farmington River, and a third development, the Hogback, is about to be started.

Nepaug Reservoir

The Nepaug Reservoir is located in the towns of Canton, Burlington and New Hartford, some twelve miles west of Hartford. It

was constructed during the 1914-1918 period by the building of Phelps Brook Dam, a 1,250-foot earth embankment on Phelps Brook, and Nepaug Dam, a 600-foot concrete masonry dam on the Nepaug River, to form an 850-acre lake of about 9.5 billion gallons capacity at flow line 482.5 m.sl. The reservoir has a 32-sq mi tributary drainage area and a 95%-year safe yield of 27 mgd.

Water is delivered from the Nepaug Reservoir to the filter plant in West Hartford through a 7½-mile, 42-inch, cast-iron pipe line, originally installed in 1913-1914, which was cleaned and lined with cement in 1944, and a paralleling 36-inch reinforced-concrete main, installed in 1941-42.

Barkhamsted Reservoir

The 32-billion gallon Barkhamsted Reservoir is located in the towns of Barkhamsted and Hartland, some eight miles north of Nepaug Reservoir and 20 miles northwest of Hartford. This reservoir stores water of the East Branch of the Farmington River, impounded by the Saville Dam, a 1,950-foot earth-fill dam, built during the 1933-39 period.

Barkhamsted Reservoir, with flow line at elevation 530 m.sl., is 8½ miles long and ⅞ of a mile wide. It drains an area of 53.8 sq mi and has a safe 95%-year yield of 55 mgd.

A 48-inch reinforced-concrete transmission main delivers Barkhamsted Reservoir water directly to the two Nepaug-West Hartford lines or, through a branch connection, directly into Nepaug Reservoir.

Compensation in Kind

Each of the Farmington Valley water-supply developments has been complicated by the fact that the Farmington River below Nepaug, the lower of the two reservoirs, is a moderately industrialized stream, with a number of mills and power plants scattered along its banks all the way to the Connecticut River, all dependent for operation to a greater or less extent on the use of water flowing in the river. To them the construction of a water-supply reservoir and its resultant diversion of water to Hartford meant a loss in use of water previously available to them in the operation of their plants. Their opposition to the original project (Nepaug) was sufficiently formida-

ble to block the first attempt to win legislative authorization for the project.

To overcome the opposition to the diversion of flow from the 32-sq mi Nepaug Reservoir watershed and to compensate for its loss the city of Hartford agreed to construct, maintain and operate for their benefit a 3-billion gallon compensating reservoir, with about 61 sq mi of free and usable drainage area, on the East Branch of the Farmington River in the towns of New Hartford and Barkhamsted, and further agreed never again to divert water from the Farmington River Valley for water-supply purposes.

This marked the beginning of a compensation-in-kind policy, which has affected each subsequent water-supply development in the Farmington River Valley.

Thus, in 1931, despite previous promises never again to tap the river for water-supply purposes, the Metropolitan District received legislative authority to develop the Barkhamsted supply by construction of the Saville Dam, at the very headwaters of the previously constructed East Branch Compensating Reservoir, with a resultant loss of flow from some 53.8 of the 61-sq mi drainage area promised under the Nepaug agreement, which essentially voided compensation for the Nepaug diversion.

To compensate for the loss of the 32-sq mi Nepaug watershed and the 53.8-sq mi Barkhamsted watershed, the Metropolitan District agreed to build, maintain and operate additional compensating reservoirs, capable of impounding 7¾ billion gallons of water, of which the existing 3-billion gallon East Branch Compensating Reservoir was to count as 2 billion gallons and the remaining 5¾-billion gallons storage was to be located on the West Branch of the Farmington River, with 20 sq mi of free and usable drainage area per billion gallons of storage.

It was further agreed that the new compensation works would be ready for service by January 1, 1955—a date which has since been extended to January 1, 1959—and, pending completion of these works, the District agreed to discharge 13.6 billion gallons of water annually (about 37 mgd) from the Barkhamsted Reservoir into the East Branch Reservoir for benefit of downstream users—an arrangement which in effect reduces the usable safe yield of Barkhamsted

Reservoir for water-supply purposes from 55 mgd to 18 mgd and the combined Nepaug and Barkhamsted Reservoirs' yield from 82 to 45 mgd.

With some 340,000 people dependent on the District for water supply, and present annual use averaging 37 mgd and increasing at an annual rate of ¾ mgd, it is essential that the present 37-mgd compensation commitment from Barkhamsted Reservoir be recaptured for general water-supply use without undue delay. To this end and to meet the compensation requirements outlined above, the District will build, maintain and operate for the benefit of downstream industries a 6½-billion gallon reservoir on the West Branch of the Farmington River.

The special law authorizing the project stipulates that the natural flow of the West Branch of the Farmington River shall not be held back at the dam, except such flows as shall be in excess of 150 cu ft per second above the dam site, and the minimum flow through the dam shall not be allowed to fall below 50 cfs, regardless of the actual natural flow of the stream.

Hogback Project

The proposed Hogback Reservoir will be formed by the construction of a dam on the West Branch of the Farmington River, at a prominent constriction in the narrow river valley about two miles upstream from the village of Riverton. The lake thus formed will have an extreme length of 4 miles, a maximum width of 0.4 miles, average depth of 36 ft (maximum 110 ft at the dam), a surface area of 560 acres at flow-line elevation 641 m.s.l., which is 111 ft higher than Barkhamsted Reservoir, and a total free usable drainage area of 104 sq mi.

Ultimately a three-mile tunnel will be constructed through the ridge separating the two valleys, making it possible to deliver about 500 mgd of West Branch flood water into the Barkhamsted Reservoir for general water-supply purposes. The proposed tunnel section is horseshoe-shaped, 7'-3" wide and 8'-2" high, with a 6- to 8-inch concrete lining, and the first 675 ft of the tunnel will be built under the Hogback Dam contract.

The Hogback project also involves the relocating and building

of a 2½-mile section of State Highway Route 8 in Colebrook, Conn., and a connecting one-mile section in Sandisfield, Mass., as replacement for a section of this highway which will lie in the proposed reservoir basin. The Connecticut section of this relocation has been completed and work is now in progress on the Massachusetts section, which is scheduled for completion this fall.

Also involved in the project was the removal and consolidation of four cemeteries in a new site off the watershed. A section of town road which crosses the dam site will be abandoned and replaced by a 4,800-foot section of road, to bypass the east end of the dam and connect with an existing road to the east. Construction of this road is included in the contract for the dam.

Hogback Dam

The principal feature of the Hogback project is, of course, the dam itself (see Figures 1 and 2).

Preliminary investigations for this structure, extending over a period of several years, have included aerial and topographical surveys, electrical observations to determine the depth to rock and character of overburden, the making of some 65 core borings at the dam site, and the collecting and laboratory testing of numerous samples of materials from upstream areas, to determine the availability and suitability of materials.

The results of these investigations indicated that the underlying rock at the dam site is schist with an average overburden of 7 ft (principally sand and gravel), and that the site is favorable for either an earth-fill or masonry dam.

The decision to build an earth dam is due to an indicated saving in cost plus the disclosure that there is an ample supply of suitable material for this type of dam within 2 to 3 miles of the site, but that suitable native material for concrete aggregate would require considerable processing to make it acceptable for use.

The principal dimensions of the dam as planned are as follows:

Length	800 ft
Maximum height above stream bed	125 ft (above rock, 135 ft)
Maximum base thickness	700 ft
Thickness at flow line	140 ft

Top width	62.5 ft
Freeboard	15 ft
Length of spillway	900 ft
Total volume of embankment	650,000 cu yd

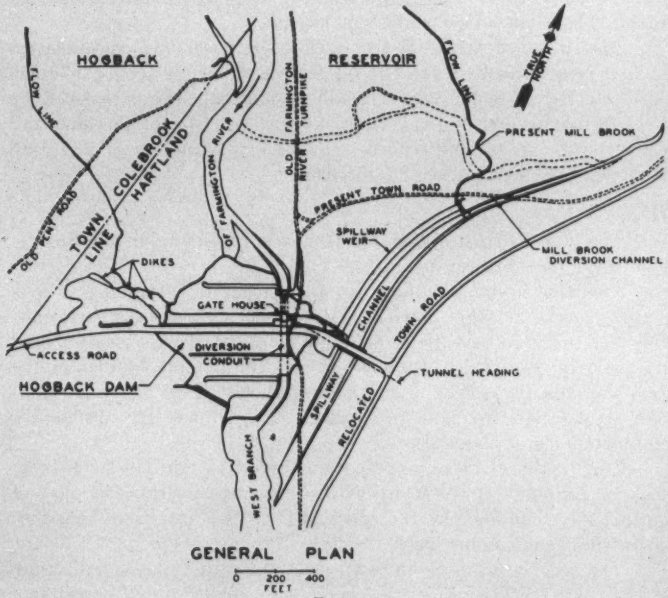


FIG. 1.

In section the dam is a combination of an earth-fill and rock-fill dam. The soil core and downstream portion of the embankment follow conventional lines of an earth-fill dam, and the upstream shell, that of a rock-fill dam.

The soil core on which the dam depends for watertightness is founded on sound rock and is centered on the center line of the dam. It has a maximum base thickness of about 100 ft and rises with 3 on 1

side-batters to a height of 12 ft above normal reservoir flow-line, where its thickness is 18 ft. The underlying rock surface will be covered with a 2-inch layer of gunite prior to the placing of soil-core material, to prevent any appreciable leakage underneath the core, and as further precaution a small concrete cut-off rib, four feet high, will underlie the soil core throughout its length.

Seams in the underlying rock foundation are to be thoroughly grouted through two lines of grout holes, 25 to 30 ft deep, located under the cut-off rib and 21 in. on either side of its center line. Grout holes are to be spaced not more than 15 ft center to center on each line and staggered.

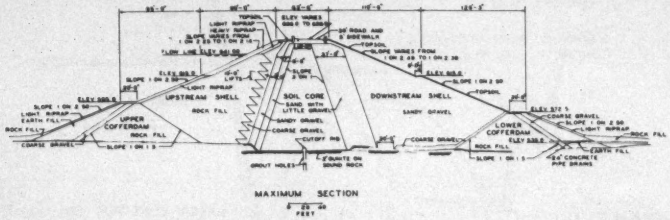


FIG. 2.

Flanking the soil core, up and down stream, are belts of sandy filtering material. The balance of the upstream shell is of rock fill, to be obtained as rock spoil from diversion-conduit and spillway-channel excavations, and the balance of the downstream shell is composed of sandy gravel material. Low cofferdams, to control the flow of the stream during early construction stages, form the upstream and downstream toes of the completed structure.

The slopes of the faces of the dam are generally 1 on 2¼ to 1 on 2½, the tops of the cofferdams forming 20-foot berms on both faces of the dam, with an additional 8-foot berm on the downstream slope, about midway between the lower berm and the top of the dam. The upstream face of the dam and the lower cofferdam will be surfaced with roughly graded riprap, and the downstream face will be covered with 12 in. of topsoil and grassed.

Extending across the top of the dam will be a 30-foot service road, paralleled by rubble parapet walls.

Diversion Conduit

The diversion conduit, a reinforced-concrete horseshoe-shaped structure, 428 ft long, 38 ft 6 in. wide and 35 ft high, with walls

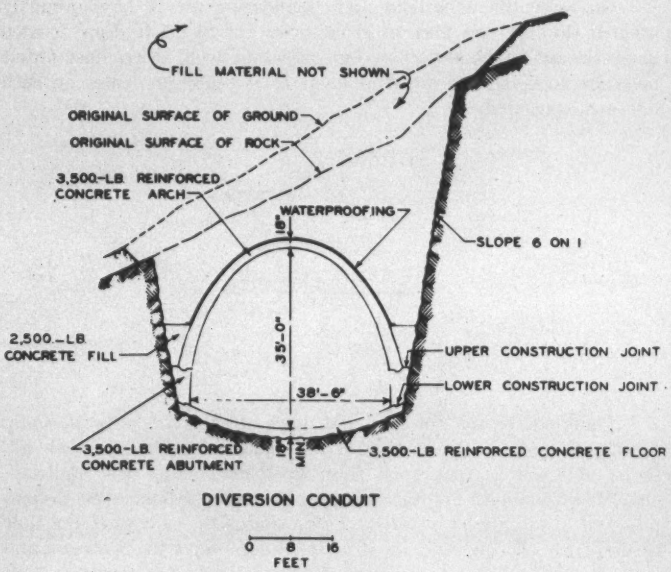


FIG. 3.

18 in. thick at the crown, is located in rock on the east side of the valley (see Fig. 3). Its construction will involve some 75,000 cu yd of rock excavation.

Its function is to handle normal and flood flows of the stream during the construction period and to discharge regulated flows for downstream users thereafter. It was designed to discharge 27,000 cfs

of water, equivalent to the maximum West Branch flood of record, without overtopping the upstream cofferdam.

Spillway and Spillway Channel

The Hogback Dam spillway weir, on the east side of the valley, is a concrete ogee-shaped structure, 900 ft long, with a 15-foot approach apron on the reservoir side (see Fig. 4). It will be securely anchored to its rock foundation, to counteract ice pressure, and the same grouting procedure will be followed for sealing the underlying rock seams as are to be used under the soil core.

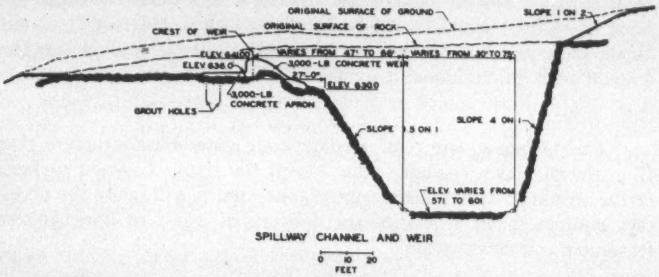


FIG. 4.

The spillway weir has been designed to discharge 92,000 cfs of water with reservoir at elevation 650 (9.0 ft above normal flow line) under a designed inflow rate of 800 second feet per square mile, as compared with 500 second feet per square mile for the Saville Dam spillway design.

The spillway channel is almost wholly in rock. It will be 1,840 ft long, about 900 ft of this being opposite the weir. The channel invert will be 72 ft wide with rock side-wall slopes of 4 on 1. The maximum depth from invert to original rock surface is 80 ft, and its construction will involve about 330,000 cu yd of rock excavation. The channel will be spanned by a reinforced-concrete arch bridge, to carry the service road across the dam.

For sanitary reasons, Mill Brook, which drains some 5½ square

miles of a cottage resort area, will be permanently diverted from the reservoir into which it would normally drain and will discharge into the upper end of the spillway channel.

Model Tests

To test the adequacy of the design of the Hogback Dam outlet works, a 1-to-50 scale model of the dam was built and tested at the Alden Hydraulic Laboratory of Worcester Polytechnic Institute.

Not only did these tests meet all design requirements (27,000 sec ft for the diversion conduit and 92,000 sec ft for the spillway weir) with very satisfactory flow performances, but further tests indicated that a flow of 143,000 sec ft (about 5¼ times the maximum flood of record) could be safely handled, with a 2¼-foot freeboard on the dam, and that the spillway channel would safely discharge this amount with an additional assumed 12,000 sec ft from Mill Brook.

Gate House

The 45-foot-square reinforced-concrete gate-house structure rises from the diversion conduit to the top of the dam. Exposed surfaces of the structure will be faced with granite, and it will house the necessary equipment for regulating the delivery of water to Barkhamsted Reservoir and downstream.

Personnel and Organization

For the Hogback project, Dr. Charles P. Berkey, former geologist at Columbia University, has acted as consultant geologist; Dr. Karl Terzaghi and Dr. Leo Casagrande of Harvard University were consultants on soils and materials of construction; and Karl R. Kennison, Chief Engineer of the New York Board of Water Supply, and Thomas H. Wiggin, Consulting Engineer of New York, have served as consultants on the review of the design and plans for the dam.

Manning W. Heard is Chairman of The Metropolitan District Commission of Hartford County; John T. Welles is Chairman of the Water Bureau, and William A. D. Wurts is District Manager. The general conduct of the work is under the direction of Warren A. Gentner, Deputy Manager and Chief Engineer of the Water Bureau, and the work of design and specifications has been done under the supervision of William Dorenbaum, Chief Designing Engineer.

CONSTRUCTION OF THE EAST TRANSMISSION MAIN IN HARTFORD AND EAST HARTFORD, CONN.

BY GILBERT U. GUSTAFSON*

[*Read April 21, 1955.*]

The East Transmission Main was the extension to East Hartford of an existing transmission main in Hartford, to provide necessary additional transmission facilities to supply East Hartford. This paper will discuss briefly the necessity for the construction of the East Transmission Main, the selection of route and pipe sizes, and some features of construction that are not usually encountered in pipe-line installations.

Prior to the construction of the East Transmission Main, East Hartford was supplied by water from Hartford through pipe-lines across the Bulkeley Bridge, augmented by a small reservoir supply in Glastonbury, yielding about 1½ mgd, which took care of Glastonbury and a small section of East Hartford.

One 16″ and two 10″ permanent lines and one temporary 16″ exposed line extend across Bulkeley Bridge. One 20″ and one 12″ line extend east from the bridge to Main St., East Hartford. Rates through these lines on extreme summer days have probably been as high as 14 mgd, with resulting pressure losses from the west or Hartford side of the Connecticut River to Main St., East Hartford, as high as 40 lb per sq in.

The State Highway Department is proposing construction of a riverfront road on the Hartford side of the Connecticut River, with a two-level intersection at the west end of Bulkeley Bridge. Maintenance of service of the water lines on Bulkeley Bridge will likely prove very difficult during periods of construction.

The severe loss of pressure, the possibility of interruption of service to the present pipe-lines, plus the undesirable feature of having the principal feeder lines all located on one bridge were the princi-

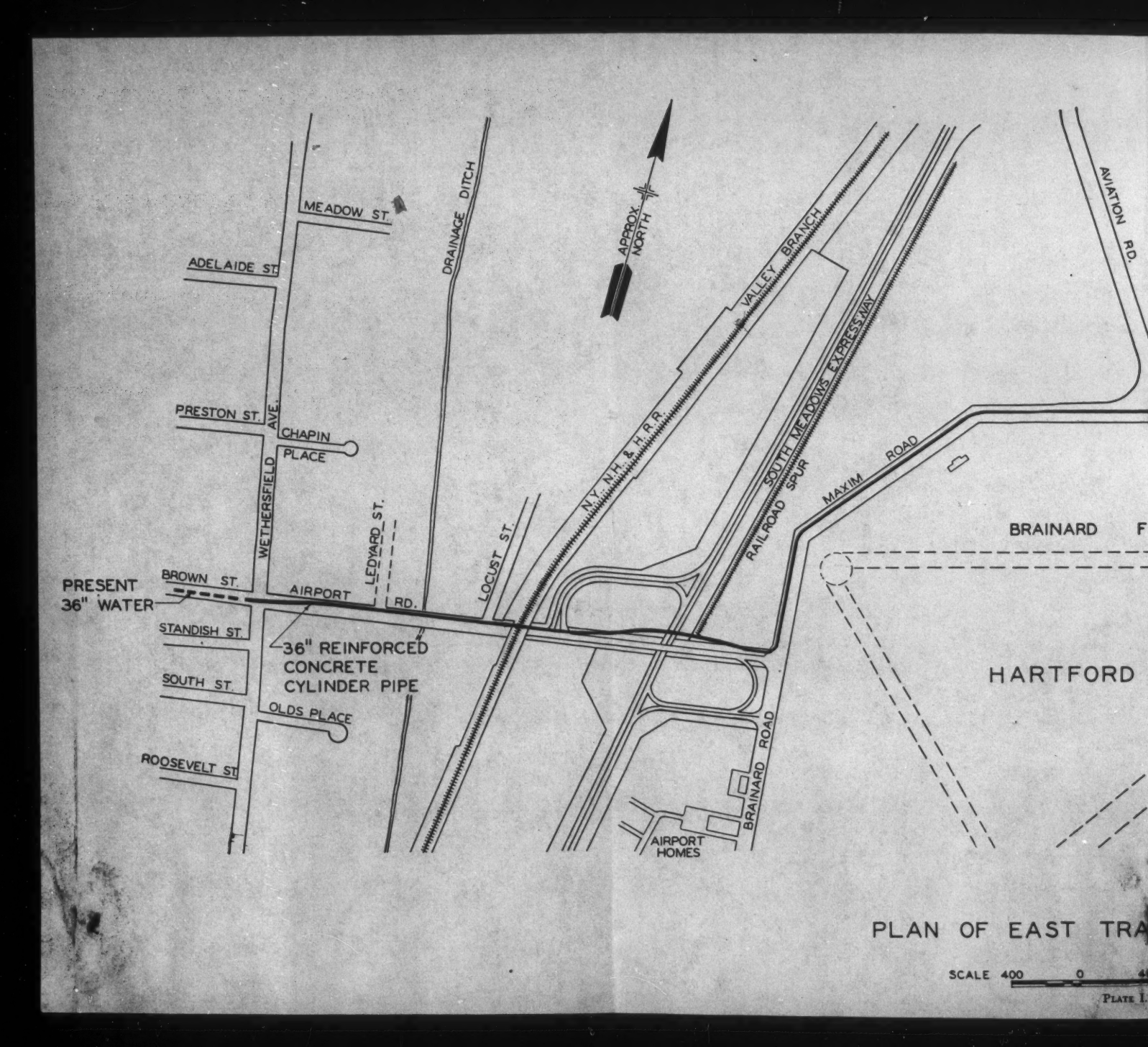
*Associate Chief Designing Engineer, Water Bureau, Metropolitan District, 115 Broad St., Hartford, Conn.

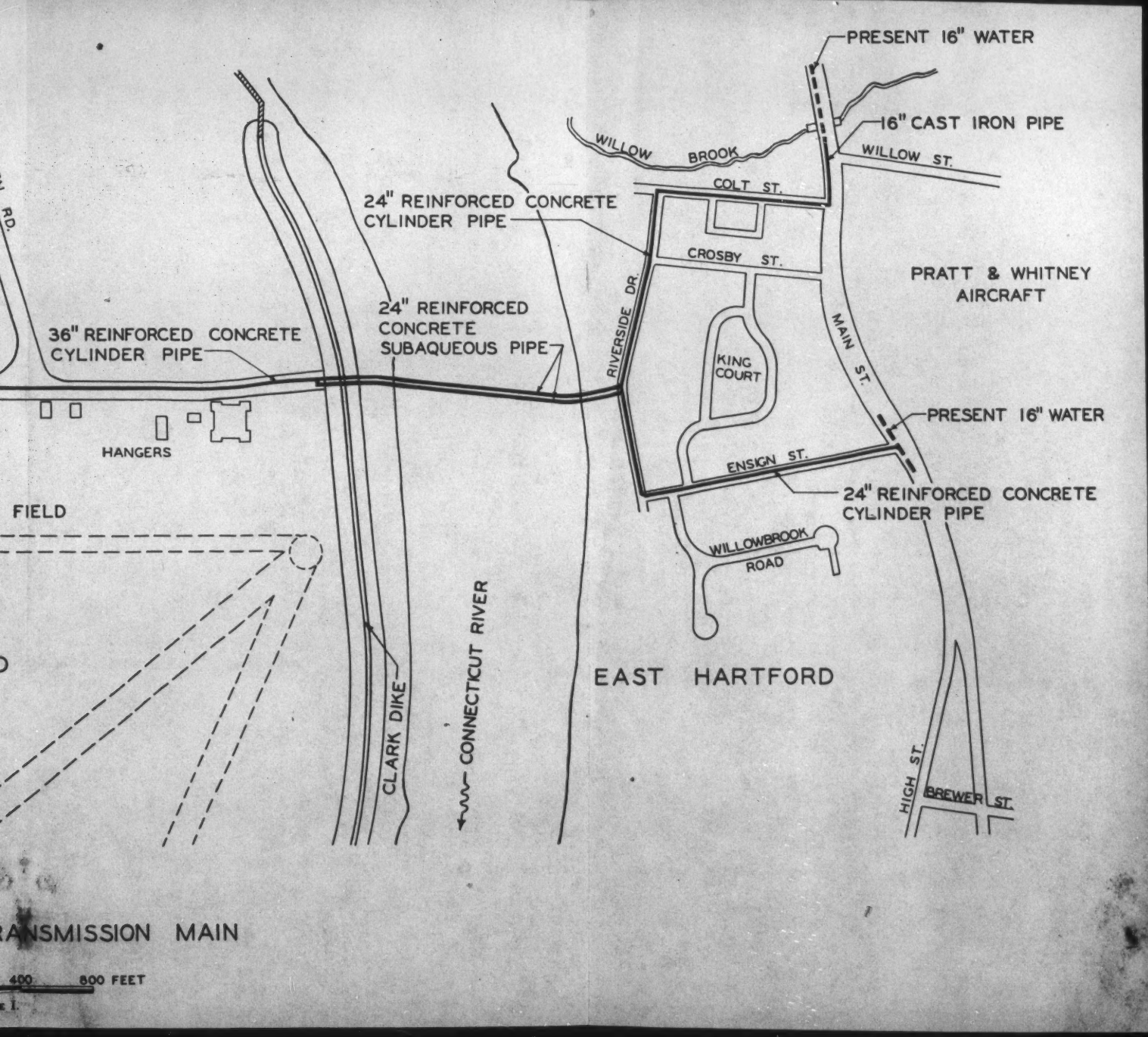
pal reasons for the construction of the East Transmission Main. Incidentally, the main plant of Pratt and Whitney Aircraft is located in East Hartford. This plant is the largest individual user of water in The Metropolitan District, the amount of use being about 14% of the District's total.

Selecting a route for the East Transmission Main was a comparatively easy task. Four to five years ago, two 5-mil gal storage reservoirs (actually a 10-mil gal basin with a division wall) were constructed on Cedar Mountain, in the southern section of Hartford, to function as balancing reservoirs. These reservoirs were tied into the existing system by the construction of large transmission mains, one of which extended east in Hartford and terminated as a 36" line at Airport Rd. and Wethersfield Ave., a natural starting point for a new line to East Hartford. In addition, the projection of a line approximately due east from this point wound up practically at the front door of the Pratt and Whitney Aircraft plant in East Hartford.

The route selected, therefore, started at Wethersfield Ave. and Airport Rd. and followed east on Airport Rd., north on Brainard Rd. and east on Maxim Rd. to Clark Dike, located on the west bank of the Connecticut River (see Plate I). Maxim Rd. is located along the north side of Brainard Field, Hartford's airport. It was considered desirable to have two lines across the Connecticut River, to insure continuity of service in case of difficulty with one of the lines, even though in the past no trouble has been experienced with the concrete pipe lines. The two-line idea was extended in East Hartford, with one line extending south on Riverside Drive and east on Ensign St. and connecting to an existing line in Main St., and the other line extending north on Riverside Drive, east on Colt St. and north on Main St. a short distance, and connecting to another existing line in Main St.

In order to provide for further growth in East Hartford, as well as the possibility of supplying Glastonbury at some future date with water from Hartford and extensions into adjacent towns, 36" was selected as the economical pipe size for the section in Hartford as far as Clark Dike and 24" for each of the river lines and sections in East Hartford, without seriously affecting pressures in East Hartford during periods of peak demands. Because of the Bureau's very


MEADOW ST.
ADELAIDE ST.
DRAINAGE DITCH
APPROX. NORTH
VALLEY BRANCH
AVIATION RD.
PRESTON ST.
AVE.
CHAPIN
PLACE
WETHERSFIELD
LEDYARD ST.
LOCUST ST.
N.Y. N.H. & H.R.R.
SOUTH MEADOWS EXPRESSWAY
RAILROAD SPUR
MAXIM ROAD
BRAINARD F
PRESENT
36" WATER
BROWN ST.
AIRPORT
RD.
STANDISH ST.
36" REINFORCED
CONCRETE
CYLINDER PIPE
SOUTH ST.
OLDS PLACE
ROOSEVELT ST.
HARTFORD
BRAINARD ROAD
AIRPORT
HOMES
PLAN OF EAST TRA
SCALE 400 0
PLATE I.


PRESENT 16" WATER
16" CAST IRON PIPE
WILLOW BROOK
WILLOW ST.
COLT ST.
24" REINFORCED CONCRETE CYLINDER PIPE
CROSBY ST.
PRATT & WHITNEY AIRCRAFT
RIVERSIDE DR.
MAIN ST.
KING COURT
36" REINFORCED CONCRETE CYLINDER PIPE
24" REINFORCED CONCRETE SUBAQUEOUS PIPE
PRESENT 16" WATER
HANGERS
ENSIGN ST.
24" REINFORCED CONCRETE CYLINDER PIPE
FIELD
WILLOWBROOK ROAD
CLARK DIKE
CONNECTICUT RIVER
EAST HARTFORD
HIGH ST.
BREWER ST.
RANSMISSION MAIN
400 800 FEET

favorable experience with concrete pipe, from both initial cost and maintenance angles, the plans and specifications were set up on the basis of reinforced-concrete cylinder pipe for the land sections and reinforced-concrete subaqueous cylinder pipe for the two lines across Clark Dike and under the Connecticut River. Both land and subaqueous pipe utilized bell-and-spigot steel joint rings and rubber gaskets. This project involved about 7,600 ft of 36″ pipe, about 3,700 ft of 24″ pipe for the land portions, and about 3,400 ft of 24″ subaqueous pipe.

The construction of the East Transmission Main was somewhat out of the routine pipe-line work, in that it involved crossing a main artery of the State Highway network, the Clark Dike and the Connecticut River.

State Highway Crossing

The Hartford section involved the crossing of the so-called South Meadows Expressway, which is the section of the Wilbur Cross Highway by-passing Hartford. This highway is very heavily travelled—particularly so during certain hours of the day, when traffic is augmented by very considerable commuter travel. Actually Airport Rd. overpasses the Expressway by bridge, but since there was no room on the bridge for a sizeable pipe, it was necessary to follow down along the toe of the Airport Rd. embankment and cross under the Expressway.

The State Highway Department would not consider an open-cut installation, unless it could be definitely proven that there was no other means of crossing the highway. Consideration, therefore, was given to jacking a pipe under the highway.

As the Lock Joint Pipe Co. advised against jacking the 36″ reinforced-concrete cylinder water pipe, consideration was given to jacking an envelope pipe and installing the concrete pipe inside. The Expressway consists of two concrete sections, each 26 ft wide, with a 10-foot dividing strip. In order to clear the shoulders, plus the fact that the crossing was on a slight skew with the highway, jacking was necessary for a distance of about 88 ft. In order to clear drainage ditches, located on either side of the highway, with adequate cover, it was necessary to locate the top of the 36″ pipe about 8 ft below the grade of the highway.

To determine at the start if jacking was feasible, borings were taken on the proposed line on either side of the highway, just outside the shoulder areas, and in the dividing strip. The samples from the borings showed a silt in the west and center holes and a fine to coarse sand in the east hole, both of which materials were suitable for jacking operations.

Because of the Bureau's previous and successful experience with jacking reinforced-concrete culvert pipe, it was decided to use the same type of pipe in this operation. A minimum inside diameter of 66″ was required, to allow space enough between the outside of the 36″ water pipe and inside of the envelope pipe, to permit installation of the water pipe and placing the jointing material in the annular recess on the outside of the water pipe. The specifications required the envelope pipe to be hand-cast, tongue-and-groove, extra-strength, reinforced-concrete culvert pipe, suitable for jacking and conforming to A.S.T.M. Specification C76, furnished in either 4-foot or 8-foot lengths.

The sheeting for the jacking pit, the backstop or reaction block, jacking frame, jacking head, guide tracks and size and number of jacks were all details left to the Contractor, subject to the approval of the Engineer. However, to minimize chances of "freezing", the Contractor was required to carry on the jacking work continuously, around-the-clock. In addition the jacking item required the installation and operation of a well-point system, to lower the ground-water level sufficiently to stabilize the material and to keep the pipe, being jacked, free of water. Since the well-points obviously could not parallel the operations across the highway, they were installed on either side of and paralleling the highway, for a distance of about 50 ft. As a matter of fact, a large part of the section of pipe-line in Hartford is located in low-lying land, known as South Meadows, with comparatively high ground water, which necessitated the installation of well-points for about 4,400 ft of pipe line, provisions for which were included in the specifications.

The 66″ envelope pipe was jacked without any unusual incidents. A steel-plate cutting edge was affixed to the outside of the front end of the forward pipe. Two men were used inside the pipe, to dig out the material as the pipe was jacked forward. The material

was loaded into a metal container, which was transported back and forth to the jacking pit by means of a small dolly. Two 250-ton hydraulic jacks with 20-inch travel were used by the Contractor. The hydraulic pressure for the jacks was supplied from one pump.

A total of about 90 hours was consumed in jacking the 88 ft of pipe, which time included all incidental operations. Eight-foot lengths of pipe were used. The line and grade of the jacked pipe finished within an inch of the line and grade shown on the contract drawings.

After the jacking operations were completed, the 36" water pipe was slid inside the envelope pipe and assembled in the usual manner, with the space between the water pipe and envelope pipe finally filled with sand, placed hydraulically.

Clark Dike Crossing

Clark Dike, at the location of the pipe-line, is an earth embankment about 25 ft high and 16 ft wide at the top, with 1 on 3½ slope on the land and 1 on 2½ on the river side. The dike was built by the Corps of Engineers, U. S. Army, but is owned and maintained by the city of Hartford. Preliminary discussions with Charles W. Cooke, Director of Public Works of the city of Hartford, revealed the fact that he looked with favor upon a pipe crossing that would follow the surface of the dike, with the pipe contained in a concrete trough, so that in the event of a pipe break, the water would tend to follow down the trough without eroding the dike.

Since two lines were already decided upon for the river crossing, it was considered desirable to carry two lines across the dike also, as a safeguard in the event of failure of a dike line during a period of high water in the river. The lines across the dike were located 25 ft apart. Preliminary borings were made on each line on the river side of the dike, to ascertain the depth of the impervious material, since it was required that construction should not penetrate the full depth of the impervious material.

The troughs consisted of U-shaped reinforced-concrete sections, 10" thick, for each line, with an inside width of 6 ft and an inside height of 4½ ft. The 24" concrete pipe was installed directly on the floor of the troughs. The troughs were then filled with sand and surfaced with top-soil on the land side and riprap on the river side,

the thought behind the sand being that it would afford protection to the pipe and yet be easily washed out in the event of a pipe break, and the escaping water would thus be confined principally in the trough.

The trough on each pipe-line ended at the top of the slope, with a return or end wall across the pipe. Each pipe-line across the top of the dike, after installation, was embedded in a concrete block. The concrete block was poured directly against the bottom and sides of the trench excavations, so that between end walls at the upper ends of the troughs there would be an effective water-stop across the top of the dike, reducing the possibility of any seepage.

At the dike, pressures in the pipe run as high as 120 lb per sq in. Instead of constructing large anchorage blocks for the bends involved, particularly the over-the-top bends, the specifications required cast-steel anchor sockets on the spring line at the ends of each pipe, to receive cast-steel lugs, with the cast-steel lugs in turn receiving steel bolts across the joint. Special steel rods were built into the pipes, running longitudinally along the pipe, welded to the anchor sockets at opposite ends of the pipe. By thus tying the lengths of pipe together, the forces tending to move the bends were balanced and the need for anchorage blocks was eliminated.

The plans for the dike crossing were approved by the city of Hartford and in turn by the Corps of Engineers, U. S. Army, who exercise control to the extent of seeing that the integrity of the dike is properly maintained.

Connecticut River Crossing

As stated previously, two lines across the river were decided upon, so that in the event of failure of one line, the second line would still be functioning. The center-to-center distance between lines was made 25 ft. Preliminary soundings were taken to determine the profile of the bed of the river, and the Corps of Engineers, U. S. Army, contacted for information on requirements and procedure necessary for a permit for the construction of the pipe-lines.

The Corps of Engineers specified that the top of the pipes be 6 ft below the 15-foot project depth of the 150-foot-wide Federal project channel, located on the east side of the river and 3 ft below

the natural bottom for the remainder of the crossing. During negotiations the Corps of Engineers changed the 150-foot width of the project channel to a 250-foot width. In order to minimize definitely the possibility of any damage to the pipe-lines, particularly during the periodic dredging of the navigable channel, the plans were prepared with the top of the pipes 8 ft below the 15-foot project depth of the navigable channel and 5 ft below the natural bottom for the remainder of the crossing.

After preparing and submitting the necessary plans, approvals for the pipe-line crossings were obtained without difficulty from the Flood Control and Water Policy Commission of the State of Connecticut and the Corps of Engineeers, U. S. Army. Permission was also obtained from the Corps of Engineers to dump material from the river trench excavations, on the west side of the river about ¾ mile downstream from the location of the pipe-line crossings.

Fourteen borings were taken in the river on a line midway between the two pipe-lines. The borings were located 100 ft center to center. These borings disclosed that, at the level of the pipe, varved clay persisted for the major portion of the crossing, with a short stretch of medium to coarse sand near the west bank.

After discussing the problem with several large contractors, experienced in marine construction work, it was decided to install the pipe on a continuous prepared bed, rather than resort to the use of piles and pile bents. Plans were prepared requiring a trench excavation a minimum of 5 ft in width at a point at least 18 in. below the bottom of the pipe, with side slopes no steeper than 3 to 1. A gravel or crushed-stone base, with aggregates ¾″ to 1½″ in size, was required, to provide a continuous bearing for each pipe. It was further required that the trenches be refilled to the present bed of the river with gravel or crushed stone, after the installation of the pipe. This specification was modified during the course of the work to require backfill only to a point 5 ft above the top of the pipe.

The 24″, reinforced concrete, subaqueous cylinder pipe has a 4″ shell thickness, as compared with 3½″ for the comparable land pipe. The details of the joint permit a 2″ joint opening in the subaqueous pipes, as compared with a 1″ joint opening in the comparable land pipe. In addition, the subaqueous pipe has cast-steel anchor sockets

on the spring line at the ends of the pipe. The anchor sockets are welded to anchorage assemblies in the pipe. Cast-steel lugs are inserted in the sockets and the lugs in turn receive steel bolts across the joint, which are used as drawbolts.

The principal floating equipment included two lighters (floating cranes), three scows (flat-top barges) and a tug. Since the lighters and scows were not equipped with power for water movements, a tug was needed on the job. The lighters were moored by cables to points on the nearest shore and to anchors in the river, marked by buoys. There was generally a scow alongside a lighter.

River traffic proved to be very annoying in performing the operations across the navigable channel. Whenever any barges, tankers or other sizeable craft approached, it was necessary to pull the lighter and scow to one side and then pull them back and realign them after the passage of such other craft. As is so often the case, or at least seemingly so, while the contractor was busy away from the channel, there was virtually no river traffic. But while operations were under way in the channel, the greater part of the day was spent in moving the lighter and scow to clear the way for the river traffic.

Excavations for the trenches were made by a lighter utilizing a clamshell bucket. The excavated material was loaded onto a scow, equipped with temporary side panels. When a scow was filled, it was brought downstream by the tug to the dumping area. Here the temporary side panels were removed, and the spoil material was pushed over the sides of the scow by means of small bulldozers, operated on the deck of the scow. When the trench excavations were to subgrade, the crushed-stone base was placed by clamshell, a scow loaded with crushed stone being used as the carrier. The crushed stone was spread by the divers by means of air jets, with minor help from the feet of the divers.

The crushed stone was trucked to the river from the quarry and chuted directly from the trucks onto the scows from a timber trestle, built on the east bank of the river.

Lines and grades for the excavation work, placing of the crushed-stone base, and installation of the pipe were set by transit and precise level from the nearest shore. The rod consisted of a comparatively

sizeable pipe, to avoid as much bending as possible from the river current. Graduations were painted on the pipe.

The diver's visibility below the surface of the water was limited to a scant few inches. Even on the brightest day, sunlight did not penetrate too deeply, because of the mud and silt in suspension. Most of the work below the water surface, therefore, was by touch alone. A phone installed in the diver's helmet was his only contact with the lighter above. The diver's phone ran to an amplifier on the lighter and could be heard all over the lighter. However, the diver could only receive messages from above when the speaker depressed a key on the phone box.

After the checking of grade and completion of the crushed-stone base, a section of pipe was lowered by the lighter crane into the river. Care was exercised in slinging the pipe before lowering, so that the lugs were exactly on the horizontal center line, to match the position of the lugs on the pipe already in place. The divers guided the pipe and, by instructions over the telephone system, advanced the pipe to engage the section previously installed. The divers, one on each side of the pipe, then engaged the drawbolts with the section previously installed and, by means of a ratchet wrench, took up on the nuts evenly, drawing the spigot into the bell until the pipe was "home". Then, to allow for flexibility in the joint, the drawbolt nuts were backed off about ¼".

During installation the pipe was checked for line and grade. If the grade of the bed proved slightly high, as was generally the case, the pipe was dropped gently once or twice; or, if slightly low, the air jet was used to reshuffle the stone to build it up. Generally only one pipe length of 16 ft was installed at a time. However, near the end of the job, two lengths were occasionally handled at a time.

After installation of a length or two of pipe, crushed-stone backfill was placed by clamshell at least to the top of the pipe. The fill was brought up evenly on either side, to insure no movement of the pipe. Later an additional foot or two of crushed-stone fill was added by clamshell, with the remainder of the fill placed by pushing the crushed stone over the side of the scow by means of the bulldozers.

At the start of the pipe-installation work in the river it was noticed that overnight settlement of the pipe was taking place. In

checking over a period of days, this settlement in some cases continued to increase, without uniformity, to a total in a few instances of 3"-4", with a resulting joint opening approaching 1".

The help of Dr. Arthur Casagrande of Harvard University was obtained, to attempt to find the reasons for the settlements. Dr. Casagrande stated that he was very familiar with the characteristics of the varved clay in this region and that it was inconceivable that the undisturbed clay below the pipe would cause any objectionable settlements due to removal and replacement of load. It was his thought that the settlement which occurred could be explained only on the basis of mud which accumulated on the bottom of the trench prior to the placement of the crushed-stone base, or mud which accumulated on top of the crushed-stone base prior to the placement of the pipe. To prevent any undue settlement, it was his recommendation to install the crushed-stone base for the pipe without delay after the excavation of the trench had been completed, and to follow this immediately with the laying of the pipe. This recommendation was followed and only minor settlements were noted thereafter, probably due to the natural rearrangement of the stone base.

The north or upstream line was completed prior to the start of the second line. One lighter was used exclusively for excavation work, excavating the trench continuously to within a foot or two of the subgrade. The other lighter excavated the last foot or two, placed the crushed-stone bed, installed the pipe and placed the backfill on the sides of the pipe, all as continuous operations for each length or two of pipe.

Leakage tests on the completed lines showed about 31% of the specified allowable leakage on the north line and about 42% on the south line. As a matter of fact, a good part of this leakage was probably through the Leadite joints in the cast-iron pipe at the gate-valve installations on either side of the river. The river lines were tested immediately after completion of installation without an ample opportunity for the Leadite to take up.

It is expected that during peak rates this summer, the pressure drop will be less than 5 lb per sq in. from Hartford to East Hartford, instead of approximately 40 lb per sq in. experienced last summer.

The contract for the construction of the East Transmission Main was awarded to American Construction Co., Inc., of Hartford, Conn., on May 6, 1954, on the basis of a bid of $704,201.75. Construction was started the last week of May, 1954, and the lines were completed and placed in service on February 8, 1955. C. W. Blakeslee & Sons, Inc., of New Haven, Conn., installed the river lines as subcontractor. The river work was started on June 24, 1954, and completed on November 30, 1954. The reinforced-concrete cylinder pipe, including the subaqueous pipe, was furnished by the Lock Joint Pipe Co.

For the Water Bureau of the Metropolitan District, the work was under the general direction of Warren A. Gentner, Deputy Manager and Chief Engineer, the supervision of the preparation of plans and specifications under William Dorenbaum, Chief Designing Engineer, and the supervision of construction under Robert J. Grosch, Associate Construction Engineer.

MEMOIR OF THEODORE L. BRISTOL

Theodore L. Bristol, Chairman of the Board of the Ansonia Water Co. (Ansonia, Conn.) and President of that company for many years, died on December 5, 1955. He was born on April 25, 1870, the son of Charles Edward Bristol and Frances Ellen Bartholomew Bristol. He was graduated from Yale in 1893. He had a long and varied experience in business affairs and held the office of director in a number of companies. His principal interest, however, centered in the Ansonia Water Co.

Mr. Bristol's activities in the water-works profession were not confined to his own company, even though he managed it ably. He was an active member of the Connecticut Water Works Association, the New England Water Works Association and the American Water Works Association. He was a former president of the Connecticut Water Works Association, and was a past president and an honorary member of the New England Water Works Association, having become a member of the latter in 1916. He greatly enjoyed attendance at meetings and was always active in discussions of water-works activities.

He not only served as a member of many water-works committees, but for many years he acted as legislative chairman for the Connecticut Water Works Association. His keen understanding of water-works problems enabled him to render invaluable service to the profession in Connecticut.

Mr. Bristol was a kindly man with a genial personality and was highly regarded by his many friends in the water-works field and elsewhere. His retirement and death were a great loss to the water-works profession and he will be greatly missed.

Mr. Bristol's wife, Florence M. Espe Bristol, died on May 10, 1952. He is survived by three daughters, Mrs. Stanley Gordon Seccombe of Oxford, Mrs. Lardner Hancock Shull of Seymour, and Mrs. William Rives Wilson of Stony Creek, as well as several grandchildren.

WARREN J. SCOTT

REPORT OF COMMITTEE ON RAINFALL AND YIELD OF DRAINAGE AREAS

RAINFALL IN NEW ENGLAND†

PART III. RAINFALL IN NEW HAMPSHIRE

ALEXANDRIA, N. H.
Record: January, 1940, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1940	2.77	3.62	5.56	5.97	6.10	6.36	4.64	1.98	3.32	0.44	6.45	3.90	51.11
1941	2.17	2.49	2.40	0.41	2.85	2.30	9.49	2.57	2.94	3.98	3.27	4.67	39.54
1942	2.36	3.76	6.75	2.95	3.01	6.06	4.67	1.37	3.77	3.37	4.78	5.78	48.63
1943	2.62	2.96	2.51	3.68	6.08	4.85	6.14	6.06	—	—	—	—	—
1944	—	—	—	—	—	—	—	—	—	3.50	3.67	3.52	—
1945	5.01	3.53	2.12	6.05	8.00	6.24	5.87	0.81	5.42	4.20*	6.36*	5.23	58.84
1946	3.28	2.92	1.77	3.49	6.73	3.48	4.99	5.55	7.34	2.32*	3.13	4.47	49.47
1947	4.11	2.46	4.77	3.33	6.69	5.77	3.22	2.32	1.40	0.84	6.07	3.38	44.36
1948	3.15	2.06	3.35	4.14	7.07	3.03	3.92	3.25	0.86	2.96	7.07	4.71	45.57
1949	5.18	2.72	2.48	3.59	3.06	4.65	3.71	3.78	4.75	3.06	3.66	1.89	42.53
1950	5.22	3.15	4.74	3.09	1.92	4.72	2.11	3.35	2.50	2.36	6.01	4.98	44.15
Av. 9 yrs.	3.69	2.97	3.77	3.67	5.05	4.73	4.74	2.78	3.59	2.61	5.20	4.33	47.13

* Interpolated.

BATH, N. H.
Record: September, 1944, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1944	—	—	—	—	—	—	—	—	4.32	3.54	1.43	2.01	—
1945	3.99	1.96*	1.49	5.84	6.42	2.26	6.42	1.95	5.94	4.16	2.89	2.80	46.12
1946	1.37	1.91	0.86	1.96	3.50	2.97	5.18	7.83	3.17	4.13	2.23	3.42	38.53
1947	3.00	1.88	2.03	1.37	5.85	5.43	3.28	0.96	1.01	0.65	3.07	1.27	29.80
1948	1.93	1.00	2.28	1.76	4.79	2.19	2.74	2.85	1.08	2.02	5.73	2.49	30.86
1949	1.84	1.79	1.25	2.30	2.88	2.95	2.59	4.41	3.77	1.85	2.74*	1.87	30.24
1950	2.91	1.74	2.36	3.01	1.68	3.74	2.36	4.70	2.54	1.84	5.05	1.72	33.65
Av. 6 yrs.	2.51	1.71	1.71	2.71	4.19	3.26	3.76	3.78	2.92	2.44	3.62	2.26	34.87

* Interpolated.

BENTON, N. H. (Glencliff)—Elevation 1,340 feet
Record: August, 1909, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	1.72	1.69	5.06	3.36	3.46	9.79	1.84	1.59	5.48	2.67	4.17	4.12	44.95
1943	1.63	1.74	1.68	3.78	4.95	5.31	5.13	6.80	2.44	6.39	4.91	0.86	45.62
1944	1.24	1.99	2.28	2.85	1.70	5.53	5.66	1.32	5.09	3.83	1.88	3.73	37.10
1945	3.77	2.24	1.73	5.51	8.00	6.30	5.49	1.21	5.52	4.73	5.96	2.22	52.68
1946	2.66	2.67	1.44	3.16	4.64	2.69	3.94	8.82	4.37	4.64	3.37	3.82	46.22
1947	3.77	2.49	2.90	1.84	6.19	7.34	6.96	1.98	1.48	0.74	5.22	1.83	42.74
1948	2.34	1.70	2.57	2.79	7.05	2.34	3.07	4.26	1.22	2.53	6.46	2.59	38.92
1949	3.33	2.15	2.49	3.21	3.60	2.55	5.09	4.26	3.93	2.83	3.13	2.60	39.27
1950	3.55	1.95	3.49	2.96	2.28	4.45	2.39	4.99	2.99	2.54	5.54	3.40	40.53
Av. 41 yrs.	2.68	2.35	3.12	3.56	3.64	4.14	4.30	3.98	4.01	3.59	3.79	2.85	42.01

† Continued from this JOURNAL 69:4, 403, December, 1955.

BERLIN, N. H. (Brown Co.)—Elevation 1,110 feet
Record: 1887 to 1903; October, 1917, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	1.52	2.75	4.77	3.45	1.88	6.16	1.53	2.07	3.54	1.64	2.88	2.84	35.03
1943	1.10	2.01	2.58	3.50	3.86	7.02	4.68	5.10	4.38	5.08	8.11	1.73	49.15
1944	2.23	2.13	1.97	2.41	1.59	5.41	3.82	1.82	4.92	2.71	3.12	3.92	36.05
1945	3.58	2.42	1.65	4.08	6.58	3.89	3.19	1.84	4.53	3.59	4.56	3.46	43.37
1946	2.95	2.25	0.75	2.19	3.13	2.98	2.83	6.96	2.65	4.53	2.19	3.90	37.31
1947	2.88	2.66	2.55	1.81	4.04	5.60	3.83	2.00	1.04	0.46	4.67	1.37	32.91
1948	1.64	1.04	1.91	1.90	4.86	2.93	2.09	1.89	0.71	2.60	4.82	2.57	28.96
1949	3.25	1.84	1.67	2.89	2.43	3.34	1.80	2.23	3.21	1.97	2.95	2.16	29.74
1950	3.09	1.85	3.58	1.88	1.40	3.92	1.82	3.77	1.87	1.80	7.26	3.28	35.52
Av. 50 yrs.	2.94	2.56	3.26	2.81	3.02	3.79	3.58	3.51	3.43	2.97	3.59	2.94	38.40

BERLIN, N. H. (Union Water Power Co.)
Record: April, 1923, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	2.07	2.20	4.95	3.27	1.88	6.18	1.64	1.96	3.56	1.62	2.88	2.85	35.06
1943	1.13	2.01	2.54	4.10	3.25	7.25	4.85	4.90	4.42	5.08	8.11	1.73	49.37
1944	2.11	2.02	1.97	2.41	2.39	4.32	3.82	1.83	4.91	2.71	4.10	3.13	35.72
1945	3.40	2.42	1.77	4.23	6.20	3.88	3.17	1.87	4.53	3.69	4.95	3.29	43.40
1946	2.82	2.09	0.87	2.19	3.85	2.26	2.84	6.96	5.28	1.88	2.36	3.92	37.32
1947	2.92	2.62	2.53	2.11	3.24	5.67	3.80	1.95	1.05	0.45	4.67	1.31	32.32
1948	1.64	1.04	1.92	2.81	3.94	2.95	2.07	1.96	0.64	2.60	4.82	3.98	30.37
1949	2.02	1.62	1.67	2.89	2.43	3.34	1.85	2.23	3.21	2.17	2.85	2.20	28.48
1950	3.21	1.86	3.45	1.88	1.40	3.93	2.48	3.44	1.53	1.72	7.26	3.28	35.44
Av. 27 yrs.	2.75	2.09	2.95	2.87	3.16	3.57	3.45	3.04	3.71	2.81	3.71	2.64	36.75

BETHLEHEM, N. H.—Elevation, 1,470 feet
Record: January, 1893, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	1.28	0.77	4.03	3.73	2.58	4.84	1.94	2.05	5.82	1.95	3.10	2.18	34.27
1943	0.65	1.03	1.64	1.92	3.40	5.33	5.52	6.05	3.12	4.87	4.93	0.59	39.05
1944	1.01	1.13	2.00	2.70	2.01	6.94	4.51	2.68	4.80	3.74	2.46	1.02	35.00
1945	1.77	0.74	0.97	4.11	6.61	3.78	4.38	1.86	5.02	4.37	4.04	2.44	39.59
1946	1.12	1.63	0.47	1.59	3.41	3.56	3.71	6.59	4.87	3.14	2.49	3.01	35.59
1947	3.59	2.00	1.78	1.25	5.86	7.77	6.09	1.91	1.43	0.60	2.19	0.55	35.02
1948	1.23	0.93	1.60	2.55	3.98	4.20	1.78	3.38	1.32	3.24	6.05	2.49	32.75
1949	1.61	0.95	1.64	2.18	4.56	2.78	2.93	3.88	5.20	2.78	2.12	2.49	33.12
1950	2.11	0.49	2.55	2.23	2.05	5.25	3.11	4.07	3.35	1.91	6.18	2.15	35.45
Av. 58 yrs.	2.01	1.95	2.56	2.58	3.03	3.83	4.02	3.51	3.79	3.02	2.90	2.33	35.53

BLACKWATER DAM, N. H.
Record: October, 1944, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1944	—	—	—	—	—	—	—	—	—	2.57	3.48	3.04	—
1945	4.01	3.51	1.57	4.67	6.23	6.65	6.74	0.89	2.20	2.60	5.76	6.24	51.07
1946	3.36	2.35	1.16	3.14	5.10	2.25	3.91	4.86	5.37	0.93	1.71	4.09	38.23
1947	4.11	2.12	3.46	3.10	3.87	5.27	5.72	1.42	2.96	0.90	5.71	2.55	41.19
1948	2.75	1.87	2.33	3.15	5.51	4.59	2.60	1.78	1.12	—	—	—	—
1949	4.97	1.96	1.48	3.25	3.42	1.69	2.28	1.12	4.77	1.92	2.70	1.28	30.84
1950	4.66*	2.83	3.40	3.02	2.10	3.63	2.11	2.63	3.15	2.87	5.11	3.90	39.41
Av. 5 yrs.	4.22	2.55	2.21	3.44	4.14	3.90	4.15	2.18	3.69	1.84	4.20	3.61	40.13

* Interpolated.

BOW, N. H. (Garvins Falls)
Record: January, 1941, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1941	2.13	1.78	1.44	0.34	2.63	1.86	3.49	1.26	1.30	1.74	3.47	3.09	24.53
1942	2.30	2.46	5.96	1.24	2.55	4.55	6.77	1.40	3.97	2.60	5.08	4.87	43.75
1943	1.95	1.59	1.45	2.94	4.65	2.20	5.48	4.16	0.90	5.15	4.77	0.46	35.70
1944	1.54	2.84	4.27	3.57	1.89	10.22	1.55	1.17	8.40	2.06	2.70	3.19	43.70
1945	3.94	3.51	1.55	4.79	5.57	5.95	4.69	2.11	1.95	2.84	4.40	5.97	47.27
1946	3.74	2.57	1.41	3.10	5.02	1.87	3.43	6.15	2.33	3.26	1.52	4.33	38.73
1947	3.25	2.31	3.47	2.97	4.06	5.69	3.75	1.70	2.91	0.71	7.33	2.29	40.44
1948	2.99	1.93	2.56	2.79	5.92	4.05	3.81	2.01	0.38	1.31	5.92	1.83	35.50
1949	4.15	2.44	1.27	2.82	4.43	0.85	2.95	1.92	5.52	2.33	3.79	1.53	34.00
1950	4.56	3.03	3.95	3.05	1.80	2.37	1.70	3.23	3.29	2.15	4.99	3.98	38.10
Av. 10 yrs.	3.06	2.45	2.73	2.76	3.85	3.96	3.79	2.51	3.10	2.41	4.40	3.15	38.17

BRADFORD, N. H.
Record: January, 1940, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1940	2.13	3.04	3.18	5.57	3.01	5.48	4.48	2.13	3.76	0.74	5.68	3.61	42.81
1941	2.46	2.65	2.43	0.36	2.17	4.19	3.38	1.50	2.27	2.30	3.19	3.53	30.43
1942	2.65	3.59	6.28	2.08	3.91	3.28	4.70	1.73	3.22	2.65	5.72	5.60	45.41
1943	2.51	1.95	2.45	—	5.38	2.75	3.44	4.95	1.13	5.74	6.75	0.57	—
1944	1.91	2.76	3.63	3.47	1.83	10.84	3.77	1.76	6.42	2.52	2.62	3.74	45.27
1945	4.83	4.00	1.84	5.54	8.36	7.09	5.17	1.39	2.60	2.81	6.76	5.10	55.49
1946	3.66	3.13	1.27	2.84	6.41	2.59	3.90	6.26	3.54	2.71	2.18	4.64	43.13
1947	4.04	2.56	4.49	2.96	4.50	4.94	5.05	1.65	1.86	0.98	6.50	3.25	42.78
1948	2.99	2.11	2.97	3.15	5.77*	3.37	2.12	2.55	0.43	2.47	5.18	2.80	35.91
1949	5.31	3.03	1.71	3.43	3.87	1.17	2.92	2.15	3.47	1.79	2.93	1.53	33.31
1950	4.56	3.15	3.69	3.35	2.24	2.92	2.44	2.83	3.63	2.86	6.32	4.80	42.79
Av. 10 yrs.	3.45	3.00	3.15	3.27	4.21	4.59	3.79	2.40	3.12	2.18	4.71	3.86	41.73

* Interpolated.

BRISTOL, N. H. (Public Service Co.)—Elevation, 430 feet
Record: February, 1940, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1940	—	3.29	4.13	6.02	4.90	4.51	4.02	—	6.70	0.50	5.61	—	—
1941	1.86	2.57	1.96	0.64	1.58	3.87	6.22	2.97	1.72	2.88	2.60	3.08	31.95
1942	Inc.	Inc.	Inc.	Inc.	Inc.	Inc.	4.41	1.23	2.61	2.21	4.92	3.51	—
1943	1.96	1.41	2.24	3.06	4.35	3.96	5.25	6.34	1.02	4.49	6.82	0.87	41.77
1944	1.58	2.91	3.42	3.10	2.13	6.77	3.47	1.14	5.79	2.93	2.85	2.47	38.56
1945	4.33	3.24	2.02	4.32	5.82	4.29	5.90	0.66	3.81	3.84	5.32	6.05	49.60
1946	3.77	2.36	1.16	2.63	4.77	2.84	4.20	5.63	7.23	1.62	2.20	3.77	42.18
1947	3.75	2.64	3.01	2.24	3.35	5.46	4.88	1.01	1.38	0.24	5.99	2.27	36.22
1948	2.81	1.51	2.91	2.29	6.27	2.10	3.13	2.81	0.47	1.95	5.27	2.56	34.08
1949	3.62	2.78	1.71	1.68	2.65	3.50	1.93	2.03	4.28	1.86	3.31	1.47	30.82
1950	—	2.48	3.70	3.22	1.44	2.92	1.23	2.77	1.13	2.37	—	3.73	—
Av. 8 yrs.	2.96	2.43	2.30	2.50	3.86	4.10	4.37	2.82	3.21	2.48	4.30	2.82	38.15

BUCKS CORNER, N. H.
Record: January, 1945, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1945	4.04	2.97	2.51	5.58	6.97	4.63	6.25	1.24	4.76	4.67	6.51	5.12	55.55
1946	2.91	2.77	1.63	3.55	5.29	3.56	3.81	5.62	3.95	1.62	2.02	4.25	40.98
1947	3.69	2.36	3.63	2.19	6.25	5.43	4.40	1.19	1.25	0.33	5.65	2.92	39.29
1948	2.99	2.07	2.92	2.70	7.06	2.32	4.03	3.39	0.98	2.38	6.02	3.93	40.79
1949	3.83	2.63	2.43	3.42	3.71	3.22	3.26	4.42	3.65	3.00	3.28	2.21	39.06
1950	4.39	3.23	4.34	2.64	1.76	2.87	2.71	3.59	3.33	2.16	5.99	3.49	40.50
Av. 6 yrs.	3.64	2.67	2.91	3.40	5.17	3.67	4.08	3.24	2.99	2.36	4.91	3.65	42.69

CAMPTON, N. H.—Elevation 660 feet
Record: January, 1941, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1941	1.77	2.62	1.48	0.62	2.69	3.24	8.61	3.29	2.38	5.08	2.24	4.08	38.10
1942	1.79	1.61	5.43	2.02	2.09	4.63	1.78	—	3.80	2.67	4.67	4.44	—
1943	1.85	1.76	2.18	3.65	4.41	5.53	5.67	4.66	2.34	4.25	5.27	0.91	42.48
1944	1.52	2.33	3.15	2.76	1.45	6.37	2.72	0.94	4.87	2.61	1.93	3.42	34.07
1945	4.06	2.98	1.88	3.62	6.18	7.07	4.78	—	4.01	4.94	4.27	4.43	—
1946	3.11	2.55	1.80	3.09	4.87	2.31	4.07	5.84	3.32	5.42	3.09	3.72	42.69
1947	3.60	2.40	3.36	1.68	4.90	4.18	5.51	1.14	0.73	0.52	5.81	1.63	35.46
1948	2.03	—	2.97	3.11	5.85	1.88	3.39	4.66	0.89	1.99	6.13	3.95	—
1949	4.59	2.26	2.36	2.82	3.46	2.85	3.41	4.42	3.66	2.57	3.79	2.47	38.66
1950	4.94	2.78	3.44	3.32	1.70	3.77	3.03	4.01	3.00	3.72	6.04	4.50	44.25
Av. 7 yrs.	3.05	2.39	2.47	2.56	3.35	4.04	4.72	3.47	2.90	3.45	4.02	2.96	39.38

CENTER HARBOR, N. H.—Elevation 740 feet
Record: January, 1945, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1945	3.20	3.75	1.87	5.04	5.67	4.45	5.31	0.85	3.82	4.93	4.61	4.98	48.48
1946	3.53	3.04	1.29	3.63	5.26	1.89	3.65	7.61	5.61	4.83	3.21	3.72	47.27
1947	3.47	1.87	4.01	2.52	5.99	4.92	2.92	2.43	1.89	0.39	6.55	2.40	39.36
1948	2.58	2.51	3.08	2.89	7.06	3.20	3.36	2.00	0.70	2.50	6.63	3.32	39.83
1949	4.59	3.17	1.61	2.60	4.17	2.30	2.23	3.55	4.98	2.42	3.03	1.78	36.43
1950	5.55	2.65	4.70	3.39	1.81	4.73	1.09	3.54	1.84	2.50	5.94	3.60	41.34
Av. 6 yrs.	3.82	2.83	2.76	3.35	4.99	3.58	3.09	3.33	3.14	2.93	5.00	3.30	42.12

CONCORD, N. H.—Elevation 350 feet
Record: 1853 to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	2.81	1.81	5.67	1.45	2.00	5.36	5.02	0.95	5.23	2.23	4.80	4.72	42.05
1943	2.12	1.57	2.47	2.82	3.89	2.35	5.18	3.19	0.73	4.33	4.38	0.58	33.61
1944	1.79	2.56	3.92	3.14	1.58	10.10	2.35	0.95	7.11	1.74	3.17	2.26	40.67
1945	3.61	3.00	1.60	4.38	5.89	6.04	4.07	1.36	1.69	2.68	3.92	5.66	43.90
1946	3.05	2.36	1.37	2.99	4.80	1.78	3.05	5.28	4.29	0.93	1.27	3.62	34.79
1947	2.92	1.93	2.98	2.98	3.37	5.03	3.82	1.18	2.50	0.59	6.68	2.10	36.08
1948	2.41	1.53	2.25	3.41	5.04	4.56	2.41	1.78	0.41	1.44	5.56	2.11	32.86
1949	4.24	1.83	1.20	2.77	3.68	1.51	3.50	2.04	4.75	2.07	2.11	1.39	31.09
1950	4.29	2.31	3.35	2.85	1.61	2.94	0.96	5.37	0.96	1.92	4.55	3.42	34.53
Av. 94 yrs.	2.99	2.63	3.14	2.96	3.13	3.34	3.64	3.48	3.38	3.09	3.35	3.00	38.13

DIXVILLE NOTCH, N. H.—Elevation 1,650 feet
Record: January, 1931, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1940	1.15	1.99	3.45	3.38	3.50	5.67	3.41	4.23	4.13	1.90	6.34	1.77	40.92
1941	1.42	1.52	2.23	0.92	2.86	4.77	4.73	4.23	2.06	4.69	2.57	1.92	33.92
1942	2.63	3.30	4.66	3.53	2.84	5.52	3.02	3.10	5.30	3.79	4.92	3.34	45.95
1943	1.04	1.87	2.56	2.34	4.33	8.21	2.84	7.74	3.50	4.43	8.47	1.61	48.94
1944	1.86	2.50	2.07	2.49	2.09	5.52	4.67	3.77	6.31	4.82	2.67	4.09	42.86
1945	4.40	1.68	2.56	4.24	8.03	4.26	5.67	3.75	8.71	5.08	3.17	1.85	53.40
1946	3.31	3.15	1.45	3.17	4.08	—	—	—	3.05	4.80	3.07	3.07	—
1947	3.54	3.29	2.61	2.50	8.04	5.34	6.75	1.82	5.12	1.33	5.00	1.14	46.48
1948	1.72	1.74	2.24	2.84	6.92	4.40	3.50	4.22	2.51	2.41	4.49	2.22	39.21
1949	2.40	1.88	2.88	2.29	4.48	6.01	3.49	4.36	5.30	3.02	5.97	2.53	42.61
1950	3.21	1.81	2.95	2.13	2.09	6.27	2.09	7.02	2.64	2.76	7.42	2.82	43.21
Av. 19 yrs.	2.87	2.22	3.15	3.14	4.08	5.02	4.70	4.11	4.68	3.54	4.12	2.65	44.28

DUBLIN, N. H.—Elevation 1,320 feet
Record: January, 1945, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1945	4.51	4.01	1.79	7.26	5.51	8.90	7.02	1.66	3.15	2.90	4.68	3.91	55.30
1946	3.06	3.30	1.27	3.33	5.63	4.32	4.43	4.91	5.23	1.40	1.20	7.00	45.08
1947	3.52	3.25	3.75	3.69	4.79	4.70	5.31	1.64	3.20	1.00	6.87	3.43	45.15
1948	2.91	2.92	3.12	2.76	7.51	5.95	4.96	3.02	0.23	2.16	6.74	3.20	45.48
1949	4.59	3.16	2.29	3.07	4.37	1.00	5.14	3.71	7.32	1.95	2.68	3.06	42.34
1950	5.35	3.70	3.41	4.16	2.79	3.82	1.74	3.39	3.11	2.15	6.86	4.51	44.99
Av. 6 yrs.	3.99	3.39	2.60	4.04	5.10	4.78	4.77	3.05	3.71	1.93	4.84	4.19	46.39

DUMMER, N. H. (Pontocook Dam)
Record: January, 1911, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	1.45	1.90	3.33	2.97	2.35	5.39	1.97	1.88	4.40	1.76	3.08	3.60	34.08
1943	1.36	1.86	2.58	3.08	3.75	6.32	3.57	5.50	3.15	3.39	5.20	1.43	41.19
1944	1.38	2.40	2.89	2.40	1.98	4.58	3.32	2.65	3.94	3.37	3.16	2.46	34.53
1945	3.82	2.86	1.41	4.37	5.19	2.44	3.31	1.78	5.27	4.06	2.90	2.62	40.03
1946	2.55	2.21	1.10	2.74	3.69	3.20	3.07	5.90	3.90	3.70	2.97	3.52	38.55
1947	3.63	2.12	2.37	2.17	4.59	5.69	5.14	2.35	1.71	0.88	3.42	1.23	35.30
1948	1.78	1.77	2.66	3.10	4.94	2.56	1.89	3.00	0.64	1.80	5.97	2.66	32.77
1949	2.46	2.24	2.08	2.34	2.78	4.53	3.25	2.22	2.68	1.89	2.39	2.91	31.77
1950	4.17	2.50	2.65	2.33	1.12	5.35	2.43	4.71	1.83	2.37	4.92	2.73	37.11
Av. 40 yrs.	2.70	2.20	2.86	2.91	2.88	3.90	3.72	3.36	3.52	3.16	3.26	2.69	37.16

DURHAM, N. H.—Elevation 88 feet
Record: January, 1893, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	2.53	2.90	8.65	2.12	1.56	3.78	5.99	1.57	2.14	2.12	4.52	6.19	44.07
1943	2.01	1.96	2.47	3.68	4.54	3.00	5.74	3.68	1.00	4.83	4.53	0.99	38.43
1944	2.48	3.46	4.43	3.84	1.08	4.24	3.39	0.89	7.21	3.12	5.95	3.58	43.67
1945	3.87	4.19	1.80	2.95	7.10	5.86	4.30	2.63	1.77	2.86	5.84	5.97	49.14
1946	3.72	3.24	1.41	3.11	4.68	2.14	2.67	8.80	3.23	1.50	1.09	4.00	39.59
1947	3.65	1.83	4.38	3.53	3.45	4.53	5.99	0.72	2.01	0.37	5.26	1.82	37.54
1948	2.80	1.82	2.35	2.51	5.04	5.43	3.54	1.95	0.46	2.46	6.10	2.42	36.88
1949	3.45	2.42	1.27	3.80	2.91	1.47	1.52	2.64	5.08	2.29	4.44	1.37	32.66
1950	4.36	2.74	4.37	2.36	1.04	2.01	1.17	5.55	2.25	3.71	6.07	3.42	39.05
Av. 57 yrs.	3.32	2.81	3.28	3.44	2.98	3.44	3.44	3.20	3.53	3.02	3.45	3.38	39.29

EASTMAN FALLS DAM, N. H.
Record: August, 1940, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1940	—	—	—	—	—	—	—	1.69	3.23	0.58	6.04	3.74	—
1941	1.66	2.15	1.44	0.41	2.79	4.46	3.90	1.84	2.01	2.72	3.20	3.20	29.78
1942	2.10	2.61	5.83	1.91	2.71	6.57	1.59	1.53	2.98	2.63	5.53	4.68	40.67
1943	2.14	1.63	2.23	3.37	5.25	2.84	5.69	6.09	0.92	4.66	6.68	0.62	42.12
1944	1.83	3.07	4.03	3.27	2.28	6.86	3.47	1.69	7.76	2.52	2.30	2.90	41.98
1945	4.64	3.42	2.02	4.97	7.14	6.81	5.06	1.11	2.14	3.49	5.30	6.26	52.36
1946	3.71	2.36	1.36	3.06	5.72	1.95	3.52	5.66	2.72	4.34	2.10	3.55	40.05
1947	3.90	1.95	3.43	2.37	4.89	4.98	4.78	1.78	2.19	0.42	5.55	2.35	38.59
1948	2.63	1.64	2.80	2.61	5.91	3.36	2.12	3.08	0.47	1.32	5.80	2.44	34.18
1949	4.18	3.01	1.33	3.01	4.34	1.10	2.34	1.34	5.14	1.97	3.01	1.49	32.26
1950	4.49*	2.69	3.75	3.73	1.83	4.67	0.82	3.46	3.11	2.62	5.22	3.90	40.29
Av. 10 yrs.	3.13	2.45	2.82	2.87	4.29	4.36	3.33	2.76	2.94	2.67	4.47	3.14	39.23

* Interpolated.

ERROL, N. H. (Errol Dam)—Elevation 1,260 feet
Record: January, 1885, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	1.79	2.06	4.85	2.47	2.03	5.63	1.68	2.25	4.57	2.34	4.08	3.41	37.16
1943	1.46	2.11	2.96	3.35	3.38	7.32	4.35	5.91	3.28	4.28	6.50	1.62	46.52
1944	1.38	2.45	2.72	2.76	1.46	4.32	5.19	3.64	4.44	4.13	3.65	2.82	38.96
1945	3.94	3.29	2.31	4.37	5.92	3.04	3.28	2.80	5.87	4.47	3.64	2.24	45.17
1946	2.85	2.64	1.37	3.30	3.89	4.27	2.99	5.35	5.04	3.14	3.44	4.73	43.01
1947	3.59	2.24	2.59	2.53	6.05	4.55	4.77	1.23	2.33	1.35	4.22	1.27	36.72
1948	2.15	1.93	2.74	2.83	5.58	2.48	1.74	2.03	1.34	2.14	6.18	3.11	34.25
1949	2.98	2.17	2.40	2.52	3.73	3.37	2.59	1.79	3.57	1.94	3.18*	2.90	33.14
1950	4.61	2.42	3.35	2.84	1.25	5.47	2.20	3.38	1.59	2.07	6.62	2.98	38.78
Av. 66 yrs.	2.83	2.48	2.85	2.76	3.14	3.84	3.82	3.70	3.51	3.06	3.30	2.88	38.17

* Interpolated.

FITZWILLIAM, N. H.—Elevation 1,200 feet
Record: January, 1920, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	2.92	1.89	5.22	1.54	3.16	3.99	5.95	2.40	3.28	3.79	3.71	4.75	42.60
1943	1.76	1.56	2.07	3.44	5.49	1.75	4.49	3.81	1.57	5.54	6.48	0.56	38.52
1944	1.71	2.35	4.05	3.18	1.13	8.65	3.91	0.74	7.03	2.36	2.22	2.56	39.89
1945	2.82	3.21	1.64	6.28	5.98	6.95	7.06	3.19	3.37	2.57	3.55	3.54	50.06
1946	2.76	2.98	1.62	2.83	6.34	3.03	3.80	5.53	4.15	1.47	1.00	3.31	38.82
1947	3.01	2.48	2.99	4.23	3.39	4.43	4.22	0.97	3.30	1.16	4.48	2.46	37.12
1948	2.53	2.22	3.58	3.93	7.07	6.06	3.61	1.97	0.49	2.22	6.13	2.41	42.22
1949	4.10	2.75	1.62	3.77	3.93	2.23	5.28	3.44	4.44	2.82	2.67	2.89	39.94
1950	4.91	2.58	2.33	3.36	3.18	4.03	3.47	7.33	1.67	2.90	7.84	4.41	48.01
Av. 31 yrs.	3.03	2.55	3.32	3.60	3.58	4.41	4.54	3.80	3.83	3.03	3.96	3.11	42.76

FRANKLIN, N. H.—Elevation 440 feet
Record: January, 1902, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	2.07	2.28	5.14	1.56	2.60	6.38	4.19	1.59	3.06	2.13	5.52	4.77	41.29
1943	2.01	1.58	2.23	3.52	4.82	2.87	5.90	6.08	0.87	4.56	6.52	0.66	41.62
1944	1.50	2.98	3.99	3.28	2.27	7.11	3.50	2.28	7.80	2.43	3.57	2.48	43.19
1945	4.72	3.27	2.14	4.58	6.41	6.62	4.89	1.16	2.13	3.65	5.59	6.65	51.81
1946	3.88	2.19	1.35	2.78	5.88	1.97	3.80	5.51	5.49	1.79	2.26	3.82	40.72
1947	4.01	2.20	4.54	2.70	4.33	5.06	4.46	1.74	2.18	0.83	6.00	2.42	40.47
1948	2.80	1.55	2.00	3.19	5.80	3.65	2.26	3.12	0.48	1.54	5.74	2.96	35.09
1949	4.03	2.50	1.41*	2.16	4.37	1.22	2.14	1.23	4.57	2.26	2.94	1.48	30.31
1950	4.38	2.17	2.72	3.08	1.87	4.52	0.93	3.81	2.68	2.71	4.76	3.89	37.52
Av. 49 yrs.	3.05	2.61	3.20	3.36	3.38	3.86	3.74	3.34	3.82	2.80	3.53	3.08	39.77

* Interpolated.

FRANKLIN FALLS DAM, N. H.—Elevation 430 feet
Record: January, 1940, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1940	2.61	4.29	3.75	5.63	5.69	3.79	3.87	2.51	3.05	0.54	6.04	2.14	43.91
1941	—	2.42	1.80	0.48	—	3.88	4.96	3.56	1.47	3.34	3.38	2.46	—
1942	2.31	1.04	5.56	1.98	2.90	6.98	4.39	2.01	3.28	—	—	5.67	—
1943	3.77	1.61	2.09	3.46	5.76	3.29	5.36	6.34	0.76	4.60	6.35	0.52	43.91
1944	1.95	3.28	3.85	3.71	1.99	7.19	2.96	1.90	8.48	2.25	3.00	2.08	42.64
1945	3.92	3.08	1.91	5.01	6.07	6.75	4.25*	0.73	1.96	3.44	5.05	6.08	48.25
1946	2.98	1.88	1.34	2.98	5.51	1.94	3.53	4.72	5.54	1.12	1.99	3.11	36.64
1947	3.95	2.04	3.30	2.78	3.89	5.64	5.31	1.80	1.78	0.79	5.33	2.16	38.77
1948	2.20	1.65	2.54	2.98	5.00	3.24	2.12	2.14	0.29	1.20	5.94	2.81	32.11
1949	4.52	2.54	1.50	2.61	4.05	0.84	2.39	1.07	4.10	2.20	2.79	1.63	30.24
1950	4.60	2.04	3.36	3.37	1.64	3.69	0.76	3.94	2.13	2.42	5.05	3.54	36.54
Av. 9 yrs.	3.39	2.49	2.63	3.61	4.40	4.04	3.39	2.79	3.12	2.06	4.62	2.67	39.21

* Interpolated.

GILMANTON, N. H.—Elevation 960 feet
Record: November, 1944, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1944	—	—	—	—	—	—	—	—	—	—	2.12	3.17	—
1945	3.18	3.37	1.78	4.15	6.22	6.05	5.58	1.31	2.48	3.36	5.12*	5.30	47.85
1946	3.51	2.02	1.37	3.31	5.31	1.43	3.40	5.39	3.89	2.86	1.82	1.97	36.28
1947	3.87	2.03	3.27	2.41	4.13	6.40	5.99	1.28	2.26	0.26	4.03	2.04	37.97
1948	2.13	1.21	2.44	2.46	5.99	3.63	1.67	1.69	0.23	1.71	6.19	2.77	32.12
1949	2.72	2.02	1.38	2.74	5.86	3.55	1.75	1.24	4.32	2.29	2.16	1.56*	31.59
1950	—	2.09	2.69	2.03	3.32	1.26	4.66	1.59	2.19	—	3.22	—	—
Av. 5 yrs.	3.08	2.13	2.04	3.01	5.50	4.21	3.68	2.18	2.64	2.10	3.86	2.73	37.16

* Interpolated.

GORHAM, N. H.
Record: January, 1911, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	1.79	2.49	4.98	3.84	1.43	5.33	1.53	2.38	4.12	2.07	3.85	4.50	37.81
1943	0.99	1.68	2.19	3.79	3.15	5.44	3.41	4.74	3.31	4.66	7.79	1.31	42.46
1944	1.79	2.05	2.26	2.42	2.05	7.00	4.08	1.72	5.40	2.85	4.68	3.99	40.29
1945	3.39	1.88	1.74	4.52	6.51	2.99	3.61	2.68	3.79	4.34	6.14	4.39	45.98
1946	2.86	2.00	1.20	2.78	3.87	2.59	2.56	7.85	5.13	2.29	2.28	3.84	39.25
1947	4.43	3.62	3.57	2.22	4.39	5.93	3.49	1.11	1.09	0.51	6.24	2.02	38.62
1948	1.79	1.49	2.61	3.42	4.76	2.44	2.36	2.31	1.08	2.33	5.61	4.77	34.97
1949	1.67	1.72	1.74	3.49	2.10	2.92	3.02	3.26	2.91	1.84	3.46	2.04	30.17
1950	3.08	1.41	4.26	2.74	1.50	4.59	2.12	4.03	1.93	2.11	8.19	5.47	41.43
Av. 40 yrs.	2.50	2.14	3.13	3.30	2.88	3.62	3.26	3.36	3.56	3.07	3.86	2.93	37.61

HANOVER, N. H. (Dartmouth College)—Elevation 603 feet
Record: 1835 to 1855 and 1867 to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	1.89	1.30	4.16	2.57	3.54	3.72	2.78	0.66	4.39	2.05	4.84	2.85	34.75
1943	2.25	1.28	1.75	3.27	4.17	3.60	4.95	5.87	1.88	4.94	5.36	0.61	39.93
1944	1.70	2.40	2.94	2.76	2.54	5.19	2.41	0.90	4.76	3.05	2.42	1.69	32.76
1945	3.53	2.56	1.59	5.02	4.88	4.17	6.06	0.70	4.08	4.37	5.30	3.23	45.49
1946	2.24	2.16	1.26	2.46	4.23	2.96	2.80	4.73	4.68	2.70	1.67	3.81	35.70
1947	3.60	2.17	2.43	1.84	5.05	4.63	3.93	1.50	0.78	0.45	4.61	2.31	33.30
1948	2.67	1.78	2.41	3.33	4.39	2.79	4.57	2.86	1.31	2.19	5.26	2.96	36.52
1949	3.07	2.02	1.70	2.16	2.39	3.07	4.74	3.08	3.62	2.47	1.85	1.94	32.11
1950	3.81	2.36	3.01	1.95	1.45	2.50	2.94	3.94	1.55	1.77	4.81	2.14	32.33
Av. 105 yrs.	2.68	2.33	2.63	2.65	3.20	3.50	3.63	3.34	3.24	3.08	2.99	2.59	35.86

HILLSBORO, N. H. (2 W)—Elevation 750 feet
Record: February, 1940, to October, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1940	—	3.67	—	5.37	5.38	—	—	1.81	—	0.71	5.13	3.72	—
1941	2.56	2.48	2.06	0.31	3.27	—	5.02	2.47	1.39	1.91	3.39	3.36	—
1942	3.36	2.23	5.99	1.31	2.80	4.49	5.53	1.82	4.19	2.34	5.06	5.44	44.56
1943	2.47	2.16	2.22	3.57	4.20	3.11	2.40	4.68	1.16	4.75	5.70	0.48	36.90
1944	1.45	—	3.40	—	1.32	10.36	3.65	2.20	6.97	1.37	2.99	2.94	—
1945	4.05	3.43	—	5.02	5.88	6.23	4.79	2.00	1.95	2.87	6.31	4.66	—
1946	3.19	3.24	1.29	2.11	5.60	2.25	4.99	4.11	4.41	0.95	1.77	4.48	38.39
1947	4.09	2.66	4.51	3.36	3.46	4.52	5.66	3.63	2.27	0.75	5.79	3.15	43.85
1948	3.37	2.49	3.27	3.79	6.59	4.10	3.57	1.87	0.41	1.95	5.46	3.45	40.32
1949	5.03	1.39	1.09	3.12	3.19	1.02	3.20	3.91	3.70	1.72	2.23	1.36	30.96
1950	4.74	3.97	3.30	2.24	3.79	0.94	5.24	1.19	2.64	6.53	—	—	—
Av. 6 yrs.	3.58	2.36	3.06	2.88	4.31	3.25	4.22	3.34	2.69	2.08	4.33	3.06	39.16

JEFFERSON, N. H.—Elevation 1,400 feet
Record: November, 1940, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1940	—	—	—	—	—	—	—	—	—	—	5.13	2.50	—
1941	1.55	3.10	1.20	0.65	3.31	2.53	6.54	3.76	3.68	3.87	1.93	1.29	33.41
1942	1.13	1.88	3.06	3.36	2.13	3.86	1.18	1.14	4.69	2.12	—	—	—
1943	1.02	1.88	1.72	1.89	3.46	5.18	3.79	5.65	3.53	4.25	5.18	1.43	38.98
1944	1.57	2.07	2.55	2.66	2.23	4.00	6.36	3.48	5.17	3.53	2.40	2.82	38.84
1945	3.24	1.55	1.27	4.18	6.89	3.47	3.67	2.48	5.29	4.34	3.98	2.53	42.89
1946	2.85	2.57	0.93	2.47	3.93	2.67	3.96	7.16	2.34	5.13	2.35	3.48	39.84
1947	3.16	2.04	2.11	1.20	4.78	7.57	5.40	2.38	1.80	1.17	3.01	1.28	35.90
1948	1.39	1.54	2.64	1.97	6.67	3.03	1.81	3.43	1.44	3.31	6.18	1.58	34.99
1949	2.52	1.91	2.33	2.58	3.83	3.98	3.44	3.70	4.08	2.49	2.34	3.06	36.26
1950	3.14	2.03	3.05	2.58	1.99	4.89	2.78	3.76	2.54	1.96	7.17	2.02	37.91
Av. 9 yrs.	2.27	2.08	1.98	2.24	4.12	4.15	4.19	3.98	3.32	3.34	3.84	2.17	37.68

KEENE, N. H.—Elevation 490 feet
Record: 1892 to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	2.15	1.79	5.40	1.42	3.94	3.74	6.23	1.87	2.96	2.62	4.75	3.81	40.62
1943	2.14	1.69	2.36	3.60	4.35	2.29	2.78	2.57	1.15	4.21	5.98	0.51	33.63
1944	1.66	2.20	4.01	3.13	1.71	6.41	3.14	1.51	6.60	1.98	2.47	2.04	36.86
1945	3.79	3.50	1.91	6.21	6.21	7.54	5.43	1.93	2.06	2.46	3.49	3.37	47.90
1946	2.33	2.21	0.98	2.27	6.30	2.83	3.08	4.06	4.50	1.60	1.22	4.50	35.88
1947	3.43	2.22	3.36	3.26	3.41	4.54	6.27	1.49	2.55	1.43	5.80	2.16	39.92
1948	2.61	2.21	2.93	3.82	5.93	6.41	3.88	2.09	0.20	1.27	5.33	2.63	39.31
1949	4.04	1.80	2.01	2.96	3.65	2.68	2.65	3.34	6.21	2.01	2.14	3.00	36.49
1950	4.82	2.79	2.25	3.49	2.30	3.08	2.29	4.82	4.50	1.51	6.87	3.04	41.76
Av. 59 yrs.	2.83	2.54	3.05	3.01	3.26	3.36	3.82	3.57	3.51	2.59	3.20	2.91	37.65

LAKEPORT, N. H.—Elevation 500 feet
Record: January, 1857, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1941	2.81	3.11	2.92	0.55*	3.26	1.95	3.81	2.39	2.52	3.24	3.73	3.54	33.33
1942	2.55	3.62	6.74	2.43	2.11	5.91	3.95	1.06	2.63	3.09	5.22	6.34	45.65
1943	1.97	1.67	2.33	3.90	4.39	3.72	7.07	5.94	0.85	5.33	6.51	0.77	44.45
1944	2.10	2.77	3.78	3.17	1.92	7.92	3.25	1.83	8.01	3.45	3.10	2.84	44.14
1945	3.90	3.09	1.89	4.00	6.69	6.02	4.68	1.00	2.17	4.56	5.00	4.89	47.89
1946	3.11	2.17	1.58	2.89	4.72	2.31	4.00	6.53	3.81	3.64	2.43	3.46	40.65
1947	4.04	2.35	3.79	2.28	3.70	6.23	2.14	0.94	2.11	1.23	6.11	2.62	37.54
1948	2.37	1.43	2.83	2.15	7.50	2.78	2.55	1.77	0.84	1.45	6.72	2.57	34.96
1949	4.54	2.38	1.41	2.82	4.48	2.45	2.10	1.47	5.26	2.20	2.71	1.71	33.53
1950	4.03	2.55	3.32	2.82	1.45*	4.06	1.06	3.31	2.68	2.07	7.29	2.88	37.52
Av. 92 yrs.	3.61	3.41	3.82	3.28	3.30	3.47	3.81	3.47	3.67	3.32	3.89	3.48	42.53

* Interpolated.

LANCASTER, N. H.—Elevation 880 feet
(Record broken as shown.)

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	—	—	—	—	—	—	—	—	—	2.01	3.72	2.84	—
1943	0.96	1.86	1.52	2.33	3.82	5.23	4.28	4.56	3.95	3.86	6.04	0.89	39.30
1944	1.12	—	2.58	—	2.17	4.81	5.50	2.99	4.48	3.72	2.60	—	—
1945	4.78	—	1.33	4.48	6.79	3.66	2.89	2.54	6.85	4.73	3.90	2.51	—
1946	2.59	2.24	1.04	2.14	—	2.31	2.63	5.56	3.34	4.51	2.98	4.17	—
1947	3.42	1.70	1.92	1.62	5.32	6.31	7.11	2.50	1.91	0.63	3.35	1.26	—
1948	2.09	1.37	2.97	1.98	5.59	4.86	2.54	—	1.43	2.67	5.96	—	—
1949	—	2.03	1.89	2.58	—	2.76	3.03	2.74	3.05	2.51	—	—	—
1950	—	—	2.78	2.19	—	4.25	3.55	3.05	1.44	2.24	—	2.14	—

LEBANON, N. H. (Airport)—Elevation 562 feet
Record: January, 1944, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1944	1.08	1.57	3.06	2.86	1.79	5.61	1.42	0.90	4.31	3.25	2.34	1.59	29.78
1945	3.20	2.28	1.50	4.68	4.80	4.99	6.01	0.40	4.96	4.22	5.35	3.41	45.80
1946	2.16	2.00	1.29	1.99	3.87	2.75	1.96	2.80	4.66	1.39	1.54	2.22	28.66
1947	—	—	—	—	—	—	3.03	1.66	1.10	0.24	2.31	3.12	—
1948	3.92	3.16	2.44	4.00	3.71	2.98	4.96	3.69	1.10	2.26	8.35	3.09	43.66
1949	4.49	2.51	1.31	1.44	2.33	1.94	2.61	2.97	3.48	1.69	1.69	1.86	28.32
1950	3.68	2.66	4.49	2.32	1.77	2.43	2.55	2.62	1.92	1.91	5.17	1.92	33.44
Av. 6 yrs.	3.09	2.36	2.35	2.88	3.04	3.45	3.25	2.23	3.40	2.45	4.07	2.35	34.92

LINCOLN, N. H.—Elevation 830 feet
Record: January, 1921, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	2.09	2.21	4.92	2.66	2.40	5.46	1.38	2.11	5.85	3.24	6.12	3.91	42.35
1943	2.17	2.07	2.80	4.73	5.57	5.40	5.44	5.93	3.17	5.18	6.02	1.34	49.82
1944	1.31	2.59	4.04	2.76	1.82	6.47	5.28	1.00	5.22	3.58	1.48	3.88	39.43
1945	5.08	3.91	2.10	5.16	6.63	4.27	3.59	1.27	7.71	5.21	4.55	4.33	53.91
1946	2.90	2.52	1.59	2.79	3.66	2.42	3.47	7.63	3.72	4.85	3.90	4.89	44.34
1947	4.72	1.83	2.86	1.93	5.59	5.89	8.51	1.49	1.25	0.43	4.35	1.45	40.30
1948	2.25	2.15	3.70	4.11	7.09	2.26	2.33	4.25	0.90	2.24	8.18	4.03	43.49
1949	4.96	3.84	2.70	2.90	3.43	2.72	2.98	4.98	4.89	2.66	3.58	3.21	42.85
1950	—	2.83	4.39	4.50	2.65	5.28	2.48	4.79	3.37	3.45	8.45	3.46	—
Av. 29 yrs.	3.52	2.72	3.46	3.59	3.75	4.31	4.37	3.90	4.39	3.79	4.82	3.59	45.71

MANCHESTER, N. H.—Elevation 171 feet
Record: January, 1875, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	3.07	1.64	6.77	1.67	2.07	4.02	7.76	1.37	3.23	2.64	4.93	5.59	44.76
1943	1.90	1.52	2.31	3.96	4.97	1.99	4.32	3.82	0.99	5.23	5.55	0.62	37.18
1944	2.04	2.99	4.62	3.89	1.78	7.07	1.87	1.65	6.39	2.35	3.52	2.92	41.07
1945	4.54	3.62	1.91	4.44	6.32	5.66	7.56	2.04	2.04	2.29	4.88	5.90	51.20
1946	3.60	2.68	1.77	3.26	5.63	2.43	3.05	6.40	4.37	0.83	1.28	3.89	39.19
1947	3.56	2.07	3.78	4.05	4.18	4.82	5.21	0.82	2.47	0.57	6.33	2.71	40.57
1948	3.08	2.23	2.92	3.83	6.63	4.56	4.22	1.62	0.15	1.59	7.07	2.50	40.40
1949	4.38	2.67	2.09	4.04	5.20	1.25	1.83	3.63	4.92	2.28	2.52	2.46	37.27
1950	5.53	3.29	3.59	3.28	—	1.90	1.34	4.69	2.32	3.08	5.52	4.22	—
Av. 75 yrs.	3.28	2.89	3.55	3.20	3.26	3.29	3.55	3.28	3.31	2.91	3.45	3.26	39.23

MASSABESIC LAKE, N. H.—Elevation 250 feet
Record: April, 1942, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	—	—	—	1.21	2.12	4.17	—	—	2.97	—	—	—	—
1943	1.93	1.35	1.93	2.69	4.41	1.63	6.63	4.85	0.87	4.41	4.62	0.77	36.09
1944	3.76	2.61	3.64	3.32	1.54	6.08	2.85	1.70	6.31	2.42	3.35	2.74	40.32
1945	3.32	3.13	1.45	3.78	5.66	5.63	8.32	1.74	2.50	2.53	3.94	4.67	46.67
1946	3.50	2.70	1.54	2.58	4.80	2.31	3.89	6.83	1.19	4.38	0.95	2.55	37.22
1947	3.06	1.73	2.38	2.99	3.42	4.30	7.07	1.03	—	0.25	5.38	1.96	—
1948	2.45	1.55	2.00	2.67	5.69	4.25	3.67	1.48	0.22	1.73	6.23	1 45	33.39
1949	3.72	2.25	1.35	3.48	4.40	1.18	1.75	2.53	5.83	2.28	1.97	1.86	32.60
1950	4.29	1.84	2.57	2.74	1.49	—	1.20	3.80	2.79	2.54	5.12	3.37	—
Av. 6 yrs.	3.11	2.26	1.98	3.09	4.42	3.51	4.52	3.19	2.82	2.96	3.51	2.34	37.71

MILAN, N. H.—Elevation 1,180 feet
Record: January, 1929, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	1.48	1.90	3.33	2.60	2.35	5.39	1.92	1.88	4.40	1.76	3.08	3.60	33.69
1943	1.36	1.86	2.58	3.08	3.75	6.32	3.57	5.50	3.15	3.39	5.20	1.43	41.19
1944	1.40	2.40	2.89	2.40	1.98	4.58	3.32	2.65	3.94	3.37	3.16	2.46	34.55
1945	3.52	2.86	1.33	4.37	5.19	2.43	3.31	1.78	5.27	4.06	2.90	2.54	39.56
1946	2.55	2.21	1.10	2.74	3.69	3.20	3.07	5.90	3.95	3.70	2.97	3.52	38.60
1947	3.63	2.12	2.37	2.17	4.59	5.69	5.14	2.35	1.71	0.88	3.42	1.23	35.30
1948	1.78	1.77	2.66	3.10	4.94	2.56	1.89	3.00	0.64	1.80	5.97	2.66	32.77
1949	2.46	2.24	2.08	2.34	2.78	4.53	3.25	2.22	2.71	1.89	2.39	2.91	31.80
1950	4.17	2.50	2.65	2.23	1.12	5.35	2.43	4.71	1.83	2.37	4.92	2.73	37.01
Av. 20 yrs.	2.86	2.11	2.87	2.95	3.29	3.93	3.59	2.98	3.35	2.89	3.28	2.77	36.87

MILFORD, N. H.—Elevation 300 feet
Record: March, 1928, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	2.07	3.44	6.86	1.65	2.06	6.15	5.77	2.72	3.31	2.65	5.37	5.48	47.53
1943	2.80	1.34	2.73	3.06	4.95	3.01	3.98	3.66	0.41	7.08	5.17	0.85	39.04
1944	1.94	2.88	3.34	3.74	1.24	8.31	2.55	0.91	6.34	2.06	3.56	3.63	40.50
1945	4.46	3.98	1.63	4.04	6.68	6.21	5.24	2.02	2.61	1.78	4.61	4.87	48.13
Av. 17 yrs.	3.57	2.91	4.20	3.81	3.08	4.44	3.24	2.90	3.76	2.86	3.81	3.69	42.27
1946	—	3.24	—	—	7.53	1.92	—	6.47	1.97	2.41	0.95	—	—
1947	4.75	—	—	—	—	—	8.34	1.20	2.29	0.87	7.92	2.42	—
1948	2.94	2.32	—	2.74	6.55	4.89	—	0.92	0.19	2.50	6.90	1.76	—
1949	3.03	3.58	2.12	3.66	4.39	0.35	1.54	1.72	6.42	1.79	2.59	1.95	33.15
1950	4.67	3.02	3.78	2.39	2.11	—	1.12	—	—	—	—	—	—

MOUNT WASHINGTON, N. H.—Elevation 6,290 feet
Record: 1859 to 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	2.29	2.83	3.74	4.71	3.61	9.44	5.08	4.56	7.70	4.98	9.58	7.75	66.27
1943	2.24	2.67	4.28	6.07	7.71	7.09	5.58	9.73	4.42	9.98	10.33	4.27	74.37
1944	3.96	2.89	2.49	3.19	2.00	10.95	6.07	4.27	8.57	3.78	5.06	6.01	59.24
1945	4.69	2.82	2.56	7.10	9.18	6.49	6.81	4.36	8.62	9.10	10.95	5.50	78.18
1946	5.02	4.34	2.15	5.19	5.35	4.27	5.43	11.43	5.74	4.65	4.59	7.03	65.19
1947	5.65	5.20	7.57	3.35	6.68	7.96	8.98	2.77	3.59	0.75	6.31	3.47	62.28
1948	2.41	4.32	3.23	5.95	5.90	5.38	3.28	5.65	3.11	4.59	9.85	5.52	59.19
1949	4.82	2.82	3.79	6.53	5.66	6.38	6.40	6.28	8.02	5.16	3.35	3.69	62.90
1950	4.90	3.27	5.04	3.88	2.78	5.51	5.09	5.72	4.97	3.78	10.76	5.17	60.87
Av. 33 yrs.	4.76	4.49	5.92	5.86	5.98	7.80	8.14	7.13	7.83	6.57	6.89	5.79	77.16

NASHUA, N. H. (Pennichuck Water Co.)—Elevation 125 feet
Record: July, 1900, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	3.58	2.39	6.71	1.39	2.56	6.90	5.43	1.84	2.83	2.32	4.28	5.59	45.82
1943	2.82	1.31	3.20	3.09	5.49	2.42	6.36	3.07	0.34	4.93	4.97	0.75	38.75
1944	2.04	2.40	3.94	3.75	1.39	7.18	3.01	1.14	7.82	2.16	3.27	3.58	41.68
1945	3.61	4.35	1.54	3.67	5.39	5.20	5.04	1.62	1.35	2.21	4.26	5.03	43.27
1946	3.87	2.66	1.79	3.08	5.74	3.45	3.10	8.08	1.17	4.44	0.71	3.55	41.64
1947	2.98	1.85	3.08	4.01	3.40	4.63	4.55	1.16	2.15	0.50	5.12	2.49	35.92
1948	3.10	1.96	2.59	2.62	6.08	5.58	3.23	1.37	0.11	2.11	6.40	1.26	36.41
1949	3.56	3.27	1.65	4.63	3.96	1.01	2.17	3.58	4.49	2.31	2.53	1.92	35.08
1950	4.63	2.92	2.59	2.74	1.35	1.55	0.75	3.33	2.20	2.96	6.58	4.14	35.74
Av. 50 yrs.	3.10	2.77	3.44	3.57	3.10	3.47	3.36	3.38	3.32	2.86	3.36	3.32	39.05

NASHUA, N. H. (1 NE)
Record: October, 1941, to August, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1941	—	—	—	—	—	—	—	—	—	1.88	2.80	3.64	—
1942	3.36	2.70	7.10	1.20	2.43	5.79	5.50	1.25	2.61	2.32	4.26	5.90	44.42
1943	2.68	1.39	3.29	3.16	4.74	2.07	5.50	2.59	0.62	4.08	4.88	0.80	35.80
1944	1.59	2.18	3.71	3.42	1.02	6.81	1.83	0.93	6.67	2.33	4.31	3.03	37.83
1945	3.64	3.56	1.59	3.64	5.20	4.75	4.16	1.72	1.12	—	—	5.16	—
1946	3.53	2.80	1.43	2.66	5.21	2.17	2.28	7.08	5.08	0.63	0.51	3.95	37.33
1947	2.93	1.79	3.55	4.02	2.90	3.15	4.15	0.70	1.88	0.28	4.93	2.53	32.81
1948	3.10	1.79	2.57	2.34	4.09	4.59	2.73	0.13	0.05	1.48	5.86	1.69	30.42
1949	3.44	2.53	1.49	4.23	3.84	0.86	1.80	2.83	3.19	2.33	2.00	1.84	30.38
1950	4.54	—	—	—	—	—	0.77	4.29	—	—	—	—	—
Av. 7 yrs.	2.95	2.17	3.31	3.00	3.46	3.63	3.40	2.22	2.87	1.92	3.82	2.82	35.57

NELSON, N. H.
Record: October, 1940, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1940	—	—	—	—	—	—	—	—	—	1.07	4.99	3.93	—
1941	2.13	1.54	1.43	0.44	4.46	3.94	5.19	1.71	2.51	2.59	2.95	2.39	31.28
1942	2.03	2.02	5.25	1.38	3.12	5.37	5.91	1.71	3.68	2.34	4.29	5.46	42.56
1943	2.84	2.56	2.14	3.49	4.15	—	3.55	4.42	1.90	4.72	5.86	0.80	—
1944	1.86	2.49	—	2.73	2.14	8.36	4.05	1.53	8.78	2.28	3.54	2.80	—
1945	3.66	3.76	2.05	6.29	6.48	10.11	6.62	1.67	2.24	3.83	4.09	3.67	54.47
1946	2.75	2.81	1.18	2.43	6.78	2.39	4.14	5.32	—	—	1.80	3.32	—
1947	3.18	2.22	4.05	3.77	3.56	5.95	5.50	1.80	2.81	1.40	5.72	2.44	42.40
1948	2.61	2.36	3.35	3.42	6.87	6.18	5.90	3.00	0.82	1.96	5.98	4.36	46.81
1949	4.47	2.28	1.94	2.86	4.14	1.99	4.04	2.41	5.59	2.21	2.73	3.04	37.70
1950	4.65	2.58	3.16	3.49	2.46	3.71	1.79	6.84	6.27	3.09	3.16	4.27	50.47
Av. 7 yrs.	3.25	2.39	3.03	3.09	4.44	5.32	4.99	2.73	3.42	2.49	4.85	3.66	43.66

NEW DURHAM, N. H.—Elevation 640 feet
Record: April, 1924, to November, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	1.86	1.62	7.35	1.60	2.74	2.98	3.32	0.82	2.95	2.14	4.20	5.59	37.17
1943	2.74	1.94	3.24	3.39	5.07	3.52	5.90	5.45	0.67	4.22	6.22	0.81	43.17
1944	1.43	2.71	3.86	3.16	1.28	6.38	2.52	1.19	7.79	2.07	4.68	3.09	40.16
1945	3.33	3.07	1.96	3.39	5.83	6.63	3.47	0.79	2.01	3.98	5.20	5.15	44.81
1946	3.63	2.58	1.14	2.84	4.40	3.26	3.82	5.96	4.07	1.06	1.85	3.64	38.25
1947	3.93	1.91	3.47	3.19	4.49	7.60	3.43	0.94	1.73	0.19	5.70	1.86	38.44
1948	2.30	1.73*	2.40	2.50	5.78	4.20	—	2.05	—	—	—	—	—
1949	—	—	1.36	2.50	3.58	1.90	3.10	1.14	3.21	1.78	2.84	1.49	—
1950	4.76	—	—	—	—	—	0.37	4.86	1.41	2.15	5.87	—	—
Av. 21 yrs.	3.51	2.96	4.02	3.18	2.91	3.57	3.56	2.65	3.53	2.73	4.27	3.33	40.22

* Interpolated.

NEWPORT, N. H.—Elevation 700 feet
Record: January, 1930, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	1.75	2.19	4.89	1.77	3.04	4.13	3.78	1.14	2.86	2.35	4.65	4.18	36.73
1943	2.32	1.45	2.15	3.52	5.87	3.04	2.01	5.55	1.56	4.57	5.30	0.53	37.87
1944	1.75	2.54	3.05	3.03	1.87	7.21	3.77	1.06	5.23	3.15	2.48	2.39	37.53
1945	3.92	3.11	1.51	5.87	5.87	3.82	5.90	0.88	3.43	3.40	5.41	3.58	46.70
1946	2.51	2.37	1.16	2.84	6.32	2.10	3.82	3.77	4.82	2.05	1.70	3.65	37.11
1947	3.37	1.98	3.34	2.19	4.93	5.02	5.67	1.06	1.37	0.40	4.61	2.85	36.79
1948	2.55	1.88	2.92	3.29	5.22	3.74	2.11	2.60	2.09	1.47	5.30	3.25	36.42
1949	3.69	2.14	1.79	3.03	3.27	2.13	2.71	2.36	5.38	1.91	1.78	1.89	32.08
1950	3.83	2.92	3.36	2.49	2.31	3.32	2.32	5.64	2.69	1.51	5.78	2.64	38.81
Av. 21 yrs.	2.99	2.21	3.15	3.32	3.45	3.73	3.76	2.96	3.50	2.50	3.48	2.71	37.76

NORTH STRATFORD, N. H.—Elevation 900 feet
Record: January, 1938, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1938	2.82	2.28	2.02	2.46	2.92	—	—	—	7.19	1.66	2.51	4.75	—
1939	2.44	2.79	2.38	4.02	1.71	4.17	2.71	3.68	3.84	5.74	0.65	3.08	37.21
1940	0.85	1.67	3.33	3.09	6.92	2.82	4.61	1.67	5.30	1.26	5.43	3.33	40.28
1941	1.66	0.90	1.78	0.93	2.98	1.98	4.00	3.26	2.02	4.49	2.58	1.77	28.35
1942	1.63	2.51	4.40	3.20	2.96	4.80	1.60	2.95	4.28	2.67	4.99	1.84	37.83
1943	0.91	1.82	1.59	2.49	3.42	7.27	2.50	6.74	3.68	3.99	4.93	1.06	40.40
1944	1.30	1.89	1.99	2.32	2.37	2.65	4.22	2.37	4.96	4.20	1.94	2.16	32.37
1945	4.28	1.84	1.71	3.77	5.02	3.64	5.23	2.62	7.93	4.64	3.16	2.76	46.60
1946	2.08	1.71	1.28	2.27	3.28	2.78	3.22	5.81	2.86	5.07	2.99	3.16	36.51
1947	3.22	2.20	1.65	1.59	6.59	4.26	8.51	2.22	2.80	1.58	3.34	1.21	39.17
1948	1.47	1.38	2.20	2.02	4.89	3.92	1.74	5.30	1.42	2.26	5.69	2.51	34.80
1949	2.88	2.07	2.18	2.32	3.46	3.01	3.86	3.17	4.02	2.31	3.10	2.85	35.23
1950	3.09	1.71	2.56	2.55	1.09	—	2.25	5.44	2.39	2.94	5.91	2.87	—
Av. 11 yrs.	2.07	1.89	2.23	2.55	3.96	3.75	3.84	3.62	3.92	3.47	3.53	2.34	37.17

NORTH VILLAGE, N. H.
Record: January, 1941, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1941	2.17	1.95	1.38	0.41	2.97	3.83	4.03	1.47	2.01	2.33	3.51	3.01	29.07
1942	1.36	3.06	4.84	2.07	2.15	4.76	6.85	3.23	4.45	2.60	4.33	5.98	45.68
1943	2.35	1.56	2.63	3.25	6.13	2.37	3.37	3.49	1.11	5.18	6.55	—	—
1944	1.81	2.32	3.71	3.37	2.52	10.60	4.06	1.84	7.86	—	3.13	3.83	—
1945	3.74	4.50	1.36	4.72	7.04	7.85	4.74	1.81	2.81	2.57	—	4.71	—
1946	3.52	3.36	1.18	2.34	5.88	3.68	4.41	6.12	2.17	3.86	0.98	3.07	40.57
1947	3.30	1.59	2.58	3.05	3.43	4.50	2.24	2.10	2.10	0.81	6.13	2.09	33.92
1948	2.13	2.46	2.40	2.58	6.50	4.95	3.79	1.84	0.39	2.35	4.78	—	—
1949	4.47	1.67	—	3.50	3.63	0.96	2.84	3.14	5.36	1.89	1.57	—	—
1950	4.17	2.65	3.03	2.36	1.86	2.48	1.31	5.52	2.31	2.09	8.49	4.11	40.38
Av. 5 yrs.	2.90	2.52	2.60	2.05	3.26	3.85	3.77	3.69	2.61	2.34	4.69	3.65	37.93

PINKHAM NOTCH, N. H.—Elevation 2,000 feet
Record: February, 1930, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	3.16	2.90	6.71	6.06	3.59	6.25	2.18	2.20	6.20	3.61	6.78	5.56	55.20
1943	2.09	3.19	3.69	4.85	6.11	6.02	6.61	7.07	3.74	8.70	14.16	2.11	68.34
1944	3.47	3.77	4.66	4.22	2.85	9.43	6.36	1.74	7.91	3.33	5.77	3.12	56.63
1945	6.34	4.54	2.43	4.96	9.86	5.10	4.15	3.59	5.58	6.75	7.41	6.11	66.82
1946	5.57	4.01	1.97	3.40	5.16	3.51	4.12	9.82	5.29	4.30	3.09	6.31	56.55
1947	5.48	5.22	5.30	3.02	6.64	8.46	6.29	1.14	1.69	0.48	7.46	2.79	53.97
1948	3.05	2.17	3.68	5.91	7.15	3.57	2.19	3.39	1.62	4.66	8.10	5.46	50.95
1949	4.15	3.21	2.76	4.82	4.34	3.96	3.20	4.47	5.65	2.97	5.37	2.65	47.55
1950	5.25	3.37	6.68	4.49	3.25	5.01	2.30	4.75	2.67	3.71	12.92	7.09	61.49
Av. 20 yrs.	4.55	3.45	5.47	4.85	4.96	4.97	4.65	4.37	5.15	4.98	5.90	4.58	57.88

PITTSBURG, N. H. (First Connecticut Lake)—Elevation 1,660 feet
Record: January, 1918, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	2.25	3.78	4.91	3.67	3.09	5.36	3.77	3.17	5.18	3.06	4.49	3.88	46.61
1943	1.55	2.94	2.80	3.36	3.86	9.37	2.58	6.20	2.93	4.62	4.83	2.33	47.37
1944	2.53	2.56	1.94	2.30	1.85	4.22	5.54	3.05	5.32	4.48	2.49	4.07	40.35
1945	4.42	2.31	2.07	4.70	5.98	3.70	7.17	2.36	8.17	5.19	3.37	2.63	52.07
1946	3.57	3.06	1.69	3.42	3.24	3.37	4.79	4.14	2.45	6.24	4.38	5.37	45.72
1947	3.96	3.67	2.61	2.50	7.93	4.65	6.71	1.61	5.25	1.44	4.92	2.04	47.29
1948	2.11	2.57	2.46	2.89	5.85	2.57	3.49	4.41	2.64	2.56	5.71	2.81	40.07
1949	3.83	2.40	3.37	3.37	4.46	4.84	3.79	2.38	3.61	2.71	3.38	3.21	41.35
1950	4.33	2.43	3.61	2.63	2.07	8.74	3.60	5.26	2.38	2.38	5.74	2.63	45.80
Av. 33 yrs.	3.09	2.51	3.05	3.28	3.97	4.60	4.75	3.97	4.79	4.28	4.20	3.18	45.67

PITTSBURG RESERVOIR, N. H.—Elevation 1,350 feet
Record: January, 1940, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1940	0.73	0.72	2.43	2.17	3.24	4.13	3.68	1.82	3.53	1.42	4.87	1.97	30.71
1941	0.99	0.82	0.36	1.32	2.62	5.67	2.82	2.69	2.20	3.29	2.54	1.48	26.80
1942	1.18	1.18	2.68	2.68	2.53	4.96	1.66	2.26	4.57	2.62	4.19	2.04	32.55
1943	1.41	2.17	2.05	2.37	2.91	9.92	2.29	4.84	3.10	3.91	4.22	1.13	40.32
1944	1.22	1.72	1.60	2.04	2.16	2.75	5.22	2.32	4.82	4.05	2.70	1.29	31.89
1945	2.75	1.46	1.82	3.81	4.67	3.92	6.18	1.22	7.54	3.89	2.48	2.01	41.75
1946	1.84	1.43	1.50	2.38	3.30	3.22	4.11	4.44	4.39	3.67	3.35	2.66	36.29
1947	3.17	1.84	1.73	1.86	6.71	5.09	6.82	1.09	5.36	1.33	3.11	1.40	39.51
1948	1.18	1.59	1.89	—	—	1.58	2.53	3.22	1.96	2.13	5.30	2.14	—
1949	1.89	1.66	2.32	2.51	3.17	4.62	3.24	2.05	2.75	2.77	2.26	2.37	31.61
1950	3.29	1.40	2.32	2.17	1.42	8.30	3.75	4.74	1.77	2.11	—	2.56	—
Av. 9 yrs.	1.69	1.44	1.83	2.35	3.48	4.92	4.00	2.53	4.23	2.99	3.30	1.82	34.58

PLYMOUTH, N. H.
Record: January, 1888, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	2.13	2.22	5.37	2.19	2.14	4.00	2.83	1.38	4.11	2.80	5.25	4.26	38.68
1943	1.35	1.90	2.34	3.93	4.27	3.72	5.74	5.28	1.80	4.41	5.19	0.85	41.28
1944	1.43	2.73	3.57	3.24	1.84	6.63	2.71	0.83	4.77	2.25	2.57	3.01	35.58
1945	4.62	2.93	1.18	4.59	5.58	5.24	4.62	0.55	3.27	4.72	4.52	5.56	47.38
Av. 58 yrs.	3.07	2.69	3.36	3.10	3.04	3.53	3.89	3.42	3.81	3.15	3.33	3.11	39.50
1946	3.30	2.06	1.56	2.83	4.98	3.24	3.64	6.22	5.99	2.17	2.16	3.29	41.44
1947	3.89	2.34	2.71	2.05	4.68	4.58	6.77	1.70	1.10	0.68	4.90	2.11	37.51
1948	2.16	0.84	2.77	2.68	5.35	2.10	2.77	4.67	—	2.16	5.78	—	—
1949	4.29*	2.39	2.89	1.95	2.48	2.31	2.66	3.89	4.83	—	4.40	1.95	—
1950	4.87	2.59	4.12	3.22	1.73	3.94	2.21	2.94	2.49	2.66	6.49	4.37	41.63

* Interpolated.

RANDOLPH, N. H.
Record: January, 1941, to September, 1949

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1941	3.54	2.07	2.06	0.67	4.64	1.84	7.96	3.74	2.65	4.29	1.87	2.34	37.67
1942	1.63	2.58	5.10	4.20	2.10	4.52	1.71	2.76	4.12	1.80	4.43	4.70	39.65
1943	1.32	1.78	2.46	3.90	3.77	5.45	3.89	5.68	4.80	6.44	9.44	0.92	49.85
1944	1.75	2.02	2.01	2.81	2.29	7.82	5.82	3.03	7.14	3.95	5.27	3.04	46.95
1945	3.78	2.82	2.37	5.23	9.24	4.90	4.07	2.84	5.12	6.50	7.65	5.65	60.17
1946	3.15	2.54	0.96	3.14	4.38	—	3.27	8.17	4.93	3.36	3.27	4.33	—
1947	3.23	3.80	4.77	2.61	6.29	7.19	4.70	1.96	1.64	0.95	6.08	1.57	44.79
1948	1.97	1.21	2.51	2.76	6.76	2.92	3.09	2.90	1.10	3.81	6.73	4.12	39.88
1949	2.06	1.68	2.17	3.14	3.94	4.16	3.63	3.14	4.78	—	—	—	—
Av. 7 yrs.	2.46	2.33	3.04	3.17	5.01	4.95	4.46	3.27	3.80	3.96	5.92	3.19	45.57

SOUTH DANBURY, N. H.—Elevation 690 feet
Record: January, 1940, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1940	2.43	3.52	4.32	5.41	2.98	5.29	3.20	1.16	4.22	0.46	5.09	3.33	41.41
1941	2.12	3.17	2.40	0.37	2.35	4.75	4.47	1.55	2.70	2.44	2.75	3.69	32.76
1942	2.00	2.99	5.45	2.05	2.17	7.79	3.90	1.57	2.24	1.79	4.66	4.92	41.53
1943	1.97	1.64	2.04	3.12	4.65	3.30	2.98	5.92	0.77	4.91	6.13	0.60	38.03
1944	1.62	2.42	2.96	3.18	2.00	6.96	3.57	1.89	5.49	3.18	2.22	3.57	39.06
1945	4.23	3.14	1.98	5.31	5.52	5.61	5.73	0.93	3.26	3.73	6.21	5.62	51.27
1946	3.25	2.53	0.65	3.07	6.01	4.12	4.28	6.48	3.78	3.02	1.99	3.99	43.17
1947	3.63	2.25	3.28	2.27	4.39	6.08	3.80	1.63	1.43	0.35	5.52	2.12	36.75
1948	2.25	1.59	2.46	2.56	6.80	3.27	3.73	2.58	1.08	2.03	5.16	2.95	36.46
1949	3.66	2.65	1.41	2.39	4.13	2.27	3.16	2.38	3.15	2.21	2.65	1.38	31.44
1950	4.03	2.42	3.83	2.31	2.14	2.56	2.65	3.64	2.52	2.68	7.02	4.01	39.81
Av. 11 yrs.	2.83	2.57	2.80	2.91	3.92	4.73	3.77	2.70	2.79	2.44	4.49	3.29	39.24

SOUTH LYNDEBORO, N. H.—Elevation 649 feet
Record: January, 1945, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1945	3.23	4.15	1.62	4.22	6.49	6.28	6.70	2.67	2.47	2.12	4.80	5.10	49.85
1946	3.63	3.53	1.61	2.84	6.10	3.14	3.35	5.92	4.41	1.07	0.90	4.14	40.64
1947	3.41	2.84	3.15	3.72	3.72	4.45	5.47	2.71	3.21	0.79	9.09	2.37	44.93
1948	2.87	1.65	2.43	3.09	6.14	4.73	3.76	0.99	0.50	2.76	6.90	2.78	38.60
1949	4.69	2.31	1.44	4.87	3.85	1.53	3.79	4.06	4.93	2.30	2.30	1.67	37.74
1950	4.97	2.92	3.36	2.10	2.17	—	1.32	5.46	1.70	2.76	5.88	4.90	—
Av. 5 yrs.	3.57	2.90	2.05	3.75	5.26	4.03	4.61	3.27	3.10	1.81	4.80	3.21	42.36

STODDARD, N. H.
Record: November, 1944, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1944	—	—	—	—	—	—	—	—	—	—	4.24	3.29	—
1945	4.73	4.76	2.00	7.38	7.30	8.44	7.25	1.80	3.17	3.11	5.79	5.50	61.23
1946	3.06	3.41	1.61	2.62	7.01	3.48	3.32	5.22	3.34	1.93	2.38	4.82	42.20
1947	4.20	2.98	7.16	3.49	4.19	5.12	3.67	1.91	2.62	0.98	7.34	3.19	46.85
1948	3.04	2.60	4.08	4.07	6.55	4.73	3.25	1.68	1.13	2.34	6.01	4.68	44.16
1949	4.97	2.20	2.11	3.81	3.71	1.15	3.96	4.31	4.42	2.29	2.57	2.70	38.20
1950	4.58	3.34	3.97	2.77	2.77	4.28	1.18	4.18	3.51	2.98	8.15	4.78	46.49
Av. 6 yrs.	4.10	3.22	3.49	4.02	5.26	4.54	3.77	3.18	3.03	2.27	5.37	4.28	46.53

SURRY MOUNTAIN DAM, N. H.—Elevation 540 feet
Record: August, 1940, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1940	—	—	—	—	—	—	—	1.77	2.47	0.81	5.64	3.89	—
1941	1.63	1.36	1.41	—	3.10	2.85	3.93	2.49	1.49	1.91	3.09	2.87	—
1942	2.67	1.59	5.52	1.47	3.91	2.72	5.81	1.63	2.93	2.70	4.44	4.12	39.51
1943	2.00	1.61	2.06	3.44	3.67	2.89	2.23	3.71	1.32	4.81	5.68	0.49	33.91
1944	1.20	2.33	4.05	3.02	1.76	6.20	3.16	1.95	6.74	1.71	2.80	1.91	36.83
1945	3.73	3.27	1.65	6.03	4.86	7.97	4.86	1.45	2.89	2.74	3.42	3.05	45.92
1946	2.20	2.62	1.16	2.02	6.00	2.25	2.57	4.27	4.69	1.36	1.09	3.62	33.85
1947	3.73	2.11	3.09	3.16	3.23	4.50	5.43	1.75	2.21	1.01	5.58	1.89	37.69
1948	2.29	2.08	2.79	3.45	5.32	4.78	4.50	2.00	0.14	1.21	5.38	2.55	36.49
1949	4.10	1.80	1.59	2.49	4.29	2.12	2.71	3.53	4.46	1.86	2.09	2.45	33.49
1950	4.49	2.51	2.74	3.28	2.30	3.25	1.23	4.50	3.90	1.28	6.92	2.37	39.27
Av. 9 yrs.	2.93	2.21	2.74	3.15	3.93	4.08	3.61	2.75	3.25	2.08	4.16	2.55	37.44

WENDELL, N. H.
Record: January, 1940, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1940	1.55	2.84	—	—	—	2.66	3.37	1.31	3.85	0.35	4.31	3.14	—
1941	1.73	2.35	1.66	—	—	4.12	4.00	1.94	1.56	1.72	1.88	3.62	—
1942	2.47	1.81	4.41	1.80	2.72	3.96	3.57	0.75	2.54	1.84	4.46	4.36	34.69
1943	2.07	1.51	1.73	2.75	5.70	2.59	3.54	4.78	1.84	4.52	4.53	0.56	36.12
1944	1.48	2.30	2.90	2.82	1.54	6.55	3.24	1.15	4.81	3.17	2.93	2.16	35.05
1945	3.58	2.73	1.65	5.64	5.25	4.08	5.34	0.74	3.17	3.06	5.64	3.81	44.69
1946	2.45	2.23	1.19	2.47	5.87	2.34	3.95	3.73	5.34	1.32	1.41	—	—
1947	—	2.02	3.92	2.64	4.13	4.99	3.56	0.93	1.69	0.31	—	2.45	—
1948	2.12	1.70	2.27	3.95	4.73	3.50	1.91	2.84	2.18	1.50	4.09	3.80	34.59
1949	3.23	1.67	1.18	1.87	2.63	1.75	1.69	2.15	3.93	1.36	1.48	1.70	24.64
1950	3.95	2.65	3.25	1.88	2.04	3.37	—	—	1.60	1.53	6.44	3.02	—
Av. 6 yrs.	2.49	1.95	2.36	3.14	3.76	3.74	3.22	2.07	3.08	2.57	3.85	2.73	34.96

WENTWORTH, N. H.
Record: September, 1944, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1944	—	—	—	—	—	—	—	—	5.64	4.07	2.69	2.31	—
1945	4.01	—	2.32	4.96	6.54	5.19	5.83	1.38	4.79	5.19	4.44	3.93	—
1946	3.75	2.97	2.30	2.73	4.53	2.70	3.49	3.73	3.66	4.02	2.47	4.05	40.40
1947	3.38	2.21	2.90	1.89	5.69	4.45	4.99	1.09	1.24	0.36	5.36	1.93	35.49
1948	2.20	1.48	2.81	2.23	5.97	2.76	3.61	3.54	1.64	2.16	5.65	3.10	37.15
1949	2.97	2.28	2.60	2.50	3.11	2.38	3.37	6.36	4.45	2.86	2.84	2.62	38.34
1950	3.97	1.76	2.77	2.44	1.72	3.41	3.41	3.41	3.31	2.18	5.19	3.02	36.59
Av. 5 yrs.	3.25	2.14	2.68	2.36	4.20	3.14	3.77	3.63	2.86	2.32	4.30	2.94	37.59

WEST LEBANON, N. H.
Record: January, 1930, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1930	2.55	0.76	4.03	0.86	3.01	3.38	3.96	2.74	1.60	1.99	2.39	0.85	28.12
1931	1.77	1.20	2.31	2.07	4.01	4.57	8.51	2.25	3.67	2.74	1.10	2.80	37.00
1932	3.31	1.81	2.88	2.64	1.39	2.58	3.52	4.78	4.42	2.94	4.89	1.06	36.22
1933	1.65	2.29	3.19	5.45	1.60	2.15	0.76	4.84	2.30	3.66	1.69	2.50	32.08
1934	2.21	1.80	2.28	4.95	2.53	4.38	4.40	1.29	4.48	2.01	3.44	3.07	36.84
1935	4.93	2.03	1.40	2.47	2.07	5.85	3.91	2.22	4.08	0.96	4.10	1.09	35.11
1936	4.69	1.91	5.20	3.40	2.43	1.72	2.89	4.12	1.38	4.99	2.05	3.64	38.42
1937	2.38	2.46	3.40	3.00	6.28	3.84	7.69	3.01	1.11	3.68	3.47	2.62	43.44
1938	3.64	1.89	1.26	2.13	1.90	2.10	6.95	4.50	9.20	2.12	2.94	4.07	42.70
1939	3.31	2.63	2.52	3.93	—	2.42	1.95	2.99	1.82	3.90	2.38	2.07	—
1940	1.12	2.53	3.56	3.55	4.60	3.30	3.50	1.00	4.21	0.72	—	3.05	—
1941	1.51	1.26	1.82	0.40	1.98	3.33	7.17	1.57	2.00	2.99	2.09	2.36	28.48
1942	1.62	1.34	3.65	2.46	3.40	3.82	3.16	1.06	3.88	2.28	4.62	3.07	34.36
1943	2.07	1.31	1.87	3.04	4.36	3.95	4.34	6.94	1.80	—	4.86	0.66	—
1944	1.42	2.03	2.95	2.73	2.42	5.50	2.40	0.89	4.40	2.86	1.59	2.30	31.49
1945	3.67	2.19	1.59	4.80	5.07	5.35	6.18	0.73	4.78	3.97	4.74	2.70	45.77
1946	2.26	2.29	1.23	2.22	3.16	2.26	2.68	4.39	3.38	2.57	1.51	3.22	31.17
1947	3.37	1.99	2.54	1.56	4.64	5.34	5.19	1.87	0.88	0.40	4.80	2.26	34.84
1948	2.65	1.86	2.08	2.89	4.06	2.70	5.71	4.49	1.22	2.07	5.18	2.76	37.67
1949	2.74	2.60	1.62	2.17	2.56	1.80	3.04	3.08	3.76	2.28	1.98	1.33	28.96
1950	3.41	2.53	2.96	2.16	1.45	2.65	2.32	2.00	2.22	1.75	4.50	1.91	29.86
Av. 18 yrs	2.79	1.90	2.58	2.69	3.00	3.52	4.47	2.77	3.26	2.57	3.17	2.42	35.14

WEST RUMNEY, N. H.—Elevation 610 feet
Record: January, 1940, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1940	1.90	2.84	4.66	4.39	5.08	3.68	3.70	1.58	1.98	1.19	6.01	2.89	39.90
1941	1.49	1.10	1.48	0.94	3.19	3.86	10.36	2.60	2.52	4.95	2.58	—	—
1942	2.14	—	5.26	2.70	2.46	10.92	2.86	—	4.05	2.77	5.38	4.14	—
1943	2.10	1.67	1.82	3.88	8.05	5.55	5.83	6.14	2.57	5.78	4.80	—	—
1944	2.31	2.69	3.68	2.88	1.85	6.80	3.59	1.37	5.83	3.45	1.82	2.60	38.87
1945	4.45	—	1.56	4.23	5.48	4.40	3.90	1.19	4.81	3.59	4.19	5.09	—
1946	4.67	3.35	1.47	3.04	4.86	4.67	3.76	5.86	4.01	3.64	2.48	4.72	46.53
1947	3.67	2.35	3.63	2.23	5.42	4.13	3.59	2.17	1.27	0.56	5.30	2.82	37.14
1948	1.92	1.95	3.03	2.99	6.02	1.82	3.34	3.24	1.12	2.38	6.15	3.59	37.55
1949	4.25	3.05	2.16	3.27	2.49	1.66	2.94	3.77	4.19	2.86	4.14	2.11	36.89
1950	4.45	3.18	4.07	2.82	1.52	2.91	2.77	5.04	2.92	2.81	6.75	3.55	42.79
Av. 7 yrs.	3.31	2.77	3.24	3.09	3.89	3.67	3.38	3.29	3.05	2.41	4.66	3.18	39.94

WEST STEWARTSTOWN, N. H.
Record: January, 1925, to December, 1945

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	1.34	2.28	3.55	2.42	2.67	6.35	2.34	2.57	4.65	2.95	3.55	2.57	37.24
1943	0.93	2.12	1.97	5.52	3.80	10.11	4.19	4.74	4.15	4.13	4.36	1.17	44.19
1944	1.14	2.18	2.01	2.41	1.73	3.40	4.43	2.07	5.35	4.49	2.05	2.13	33.39
1945	4.13	2.36	1.92	3.80	5.07	2.54	6.59	1.87	7.78	3.64	2.95	2.24	44.89
Av. 18 yrs.	2.24	1.88	2.11	2.51	3.48	4.33	4.47	3.40	4.38	3.70	3.00	2.30	37.80

WINCHESTER, N. H.—Elevation 174 feet
Record: January, 1940, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1940	1.99	—	4.74	4.71	4.44	2.82	2.76	1.26	1.77	0.96	5.56	3.34	—
1941	2.14	1.79	1.74	0.49	2.13	2.57	6.82	2.80	1.69	1.75	3.21	3.40	30.53
1942	2.85	1.58	5.52	1.18	3.32	1.82	5.74	1.21	3.70	2.62	5.28	4.91	39.73
1943	2.33	1.63	2.77	1.51	4.24	1.10	2.05	2.20	1.75	4.55	5.19	0.57	29.78
1944	1.44	1.98	4.15	3.15	0.77	5.04	5.05	1.20	7.59	2.01	3.11	2.31	37.80
1945	3.80	3.46	2.29	5.97	6.32	6.19	6.19	1.66	1.94	2.45	3.81	3.96	48.04
1946	2.66	3.12	0.98	2.09	5.56	3.54	4.15	6.13	4.35	1.33	0.85	4.25	39.01
1947	4.05	2.16	3.11	3.13	3.23	3.48	3.90	0.91	3.26	2.12	4.89	2.56	36.80
1948	2.30	2.56	2.75	3.11	6.70	6.16	2.22	1.80	0.30	1.21	5.38	2.57	37.06
1949	4.32	2.81	1.46	3.12	3.24	1.44	2.67	1.97	5.45	2.37	1.93	2.71	33.49
1950	5.38	3.61	2.38	3.93	2.64	3.42	2.39	6.56	1.69	1.80	6.07	3.88	43.75
Av. 10 yrs.	3.13	2.47	2.72	2.75	3.82	3.48	4.12	2.64	3.17	2.22	3.97	3.11	37.60

WOLFEBORO FALLS, N. H.—Elevation 520 feet
Record: May, 1931, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	2.15	2.92	5.95	1.88	1.91	2.92	3.46	1.64	2.36	2.23	4.72	5.77	37.91
1943	2.44	2.11	2.35	4.10	5.35	3.89	6.34	3.52	0.63	3.66	6.04	0.84	41.27
1944	1.71	2.91	4.10	3.06	1.21	6.70	3.03	1.28	7.28	2.43	4.14	3.09	40.94
1945	4.35	3.61	2.00	3.53	7.24	6.68	3.48	0.79	2.15	4.05	4.91	5.28	48.07
1946	3.63	2.61	1.54	3.07	4.55	3.05	3.62	6.40	4.61	1.69	2.18	3.84	40.79
1947	4.66	2.25	3.91	2.54	5.33	6.23	3.72	0.83	2.54	0.19	4.76	2.70	39.66
1948	2.92	1.81	2.97	2.61	7.39	2.90	2.58	2.39	0.35	2.23	6.91	4.39	39.45
1949	4.88	2.56	1.93	3.32	3.31	2.94	1.87	1.73	3.47	1.85	3.54	1.75	33.15
1950	5.10	2.68	4.47	2.59	0.97	1.82	0.68	4.48	2.29	2.18	7.55	4.23	39.04
Av. 17 yrs.	3.87	2.87	3.60	3.33	3.62	3.64	3.28	2.62	3.48	2.52	4.71	3.58	41.12

WOODSTOCK, N. H.—Elevation 720 feet
Record: January, 1941, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1941	1.30	2.18	1.37	0.62	2.34	2.55	5.66	3.95	1.75	5.70	2.52	3.53	33.47
1942	2.37	1.92	4.49	2.61	2.73	5.19	1.81	1.45	6.34	3.17	6.06	3.49	41.63
1943	1.52	1.70	2.25	4.72	4.29	—	6.91	6.82	3.10	4.77	6.35	0.96	—
1944	1.09	2.71	4.03	3.06	1.67	5.32	4.26	0.94	5.72	3.59	2.38	2.25	37.02
1945	4.08	3.27	1.87	4.45	6.71	5.77	6.24	0.54	5.89	6.23	4.43	4.63	54.11
1946	3.08	2.46	1.64	3.57	4.92	2.57	3.71	7.33	5.73	3.15	3.99	4.93	47.08
1947	5.39	2.08	3.29	2.83	6.03	5.82	8.61	2.26	1.35	0.94	4.35	1.38	44.33
1948	2.05	1.84	2.82	4.49	6.82	2.44	3.28	4.17	1.41	2.24	8.54	4.57	44.67
1949	4.75	3.19	2.73	3.09	3.85	3.62	4.70	4.40	5.06	2.76	3.87	2.62	44.55
1950	5.36	2.66	4.11	4.01	2.00	5.16	3.45	4.09	3.07	3.93	8.01	3.34	49.19
Av. 9 yrs.	3.27	2.48	2.93	3.18	4.12	4.27	4.63	3.24	4.03	3.52	4.91	3.42	44.00

YORK POND, N. H.—Elevation 1,500 feet
Record: February, 1931, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	1.31	2.37	4.94	4.86	3.04	4.09	1.69	1.86	5.21	2.27	3.24	4.12	39.00
1943	1.51	2.23	2.60	2.93	3.63	—	4.54	5.36	3.73	4.36	7.48	1.33	—
1944	1.97	2.19	2.83	2.61	2.68	5.95	5.43	2.48	5.40	3.68	2.14	4.10	41.46
1945	3.62	3.01	2.52	4.16	7.31	3.28	3.62	2.49	5.48	4.40	5.06	3.26	48.20
1946	3.20	2.92	1.59	2.84	3.97	2.87	3.84	6.79	2.86	5.08	2.93	4.64	43.53
1947	3.92	2.58	5.07	1.89	5.47	6.97	4.98	3.18	2.15	1.09	4.15	1.56	43.01
1948	2.38	1.83	2.79	3.17	6.11	4.15	2.21	3.15	1.12	2.69	6.76	2.96	39.32
1949	4.53	2.47	2.17	3.41	2.75	3.70	2.43	2.60	4.03	2.59	3.20	3.37	37.25
1950	4.20	2.69	3.52	2.40	1.65	5.20	3.08	4.27	2.59	2.07	6.92	2.58	41.17
Av. 18 yrs.	3.42	2.60	3.72	3.51	3.85	4.36	4.19	3.38	4.20	3.32	3.84	3.26	43.65

PROCEEDINGS

NOVEMBER 1955 MEETING

HOTEL STATLER, BOSTON, MASS.

THURSDAY, NOVEMBER 17, 1955

President Allan F. McAlary in the Chair.

Secretary Knox announced the election of the following to membership in the Association:

Joseph M. Hackett, Chairman, Board of Water Commissioners, Hanover, Mass.; Matthew E. Kearns, Jr., Manager, Bernon LeMoine Water Company, Kingston, R. I.; William G. Wilkie, Assistant Water Works Engineer, Schenectady, N. Y.; Richard F. Mason, Utility Engineer, Maine Public Water Utilities Commission, Augusta, Me.; Paul C. Ross, Senior Sanitary Engineer, Massachusetts Department of Public Health, Boston, Mass.; John K. Carr, Vice-President, Tekton Associates, Inc., Consulting Engineers, Natick, Mass.; Robert Veo, Sanitary Engineer, Tekton Associates, Inc., Consulting Engineers, Natick, Mass.

A paper on "Radar in Flood Forecasts and Warnings" was read by William E. Hiatt, Chief, Hydrologic Services Division, U. S. Weather Bureau, Washington, D. C.

A paper on "Floods of August, 1955, in New England" was read by Harvey B. Kinnison, District Engineer, U. S. Geological Survey, Boston, Mass.

A paper on "Experiences with Public Water-Supply Damage in the Connecticut Flood Disaster of 1955" was read by Warren J. Scott, Director, Bureau of Sanitary Engineering, State Department of Health, Hartford, Conn.

DECEMBER 1955 MEETING

HOTEL STATLER, BOSTON, MASS.

THURSDAY, DECEMBER 15, 1955

President Allan F. McAlary in the Chair.

The President announced the election of the following to membership in the Association:

John B. Donohoe, Sales Engineer, A. P. Smith Manufacturing Co., Springfield, Mass.; Edward H. Haley, Chairman, Board of Water Commissioners, Winthrop, Mass.; George H. Hill, Member, Board of Water Commissioners, Winthrop, Mass.; Gerald H. Lamprey, Sales Engineer, B-I-F Industries, Manchester, Me.; Karl H. Schamberger, Engineer, Bureau of Water Supply, Baltimore, Maryland; Chester R. Spinney, Member, Board of Water Commissioners, Winthrop, Mass.

Associate Member: Walworth Co., Boston, Mass.

PRESIDENT MCALARY. Since our last meeting three members of the Association have passed away, two of them having been Honorary Members:

Theodore L. Bristol of Ansonia, Conn.
Stephen H. Taylor of Saco, Me.
James A. Newlands, of Hartford, Conn.

Let us stand in silence for a moment out of respect to their memory.

[*The members stand in silence.*]

Commander John J. Albers, Engineer Corps, United States Navy, Boston, Mass., gave an illustrated talk on the "Texas Tower."

Kenneth J. Carl, Engineer, National Board of Fire Underwriters, New York, N. Y., gave a talk on "Water Supply and Municipal Fire-Protection Classifications."

JANUARY 1956 MEETING

HOTEL STATLER, BOSTON, MASS.

THURSDAY, JANUARY 19, 1956

President Allan F. McAlary in the Chair.

Secretary Knox announced the election of the following to membership in the Association:

Harrison R. Medding, Jr., Superintendent, Water Department, Warner Village Fire District, Warner, N. H.; Ernest J. Sullivan, Sanitary Engineer, Massachusetts Department of Public Health, Boston, Mass.; William D. Monie, Chief Engineer, Portland Water District, Portland, Me.; Howard W. Anderson, Superintendent, North Kingstown Water Works, North Kingstown, R. I.

A paper on "Laying of Subaqueous Pipe Line" was read by William Dexter, Project Manager, Verrier Construction Co., Portland, Me. The paper was illustrated by slides and moving pictures.

William A. Tompkins, Chief Aquatic Biologist, Massachusetts Division of Fisheries and Game, Westborough, Mass., gave a talk on "Fishery Management in Relation to Water Supplies." Moving pictures were shown.

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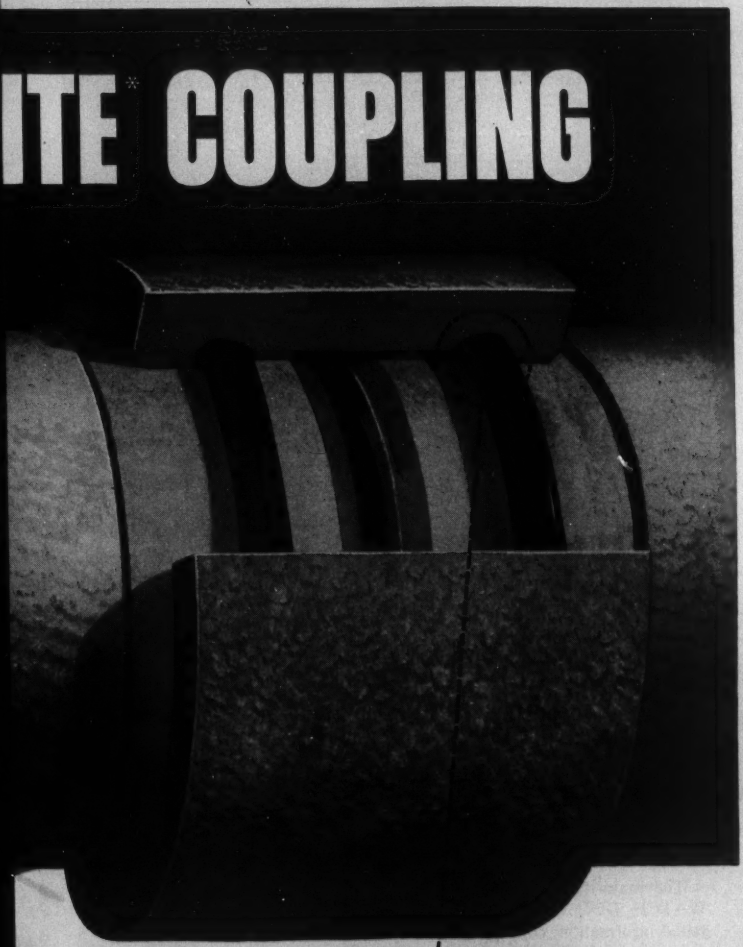

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pipe to slide into coup-
easily. Water pressure
natically expands the
The higher the internal
pressure, the tighter
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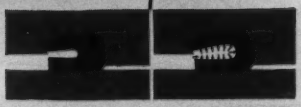

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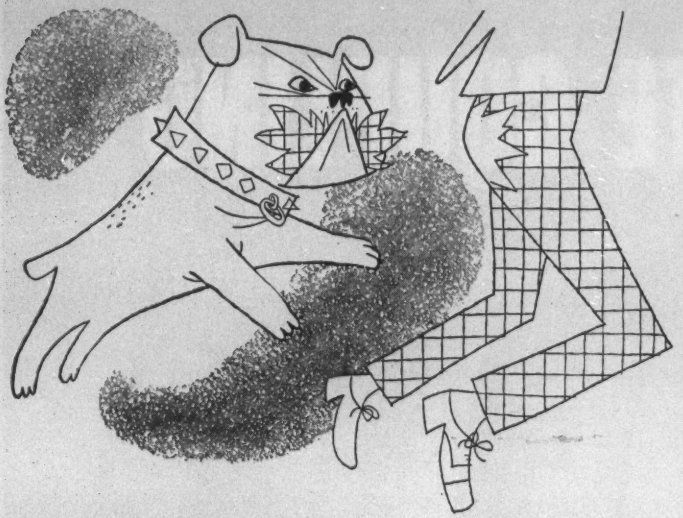

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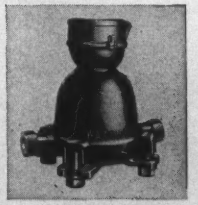

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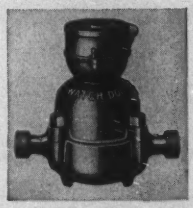

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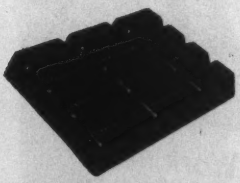

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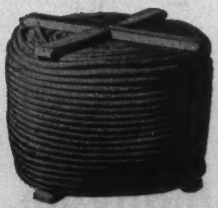

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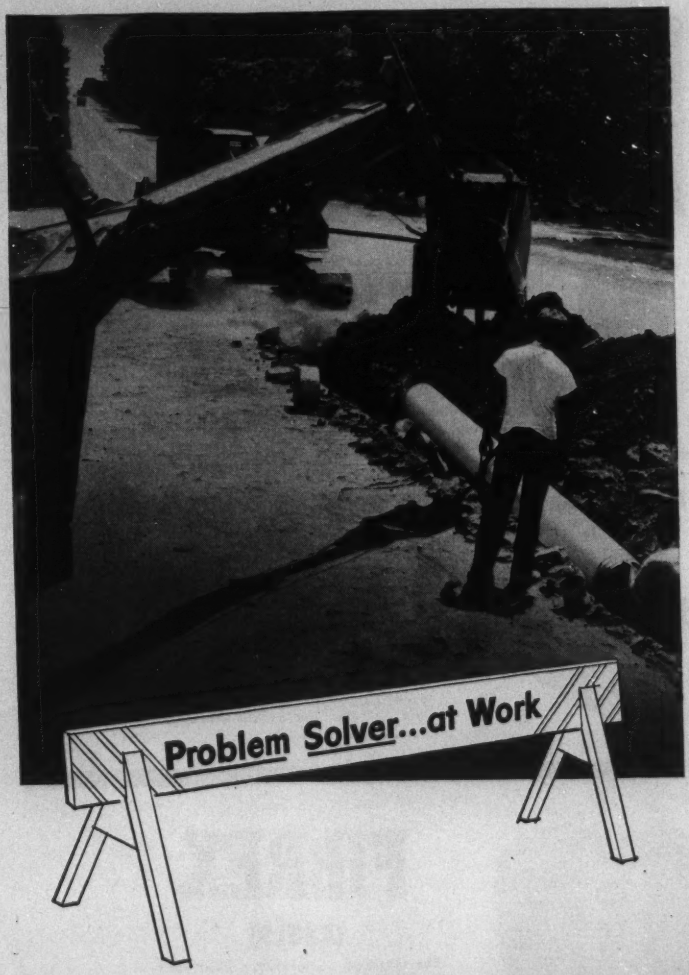

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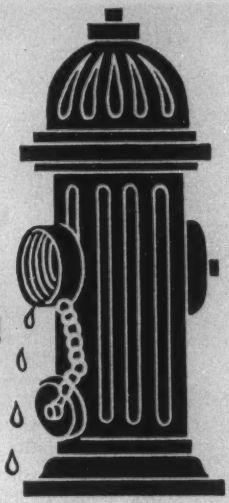

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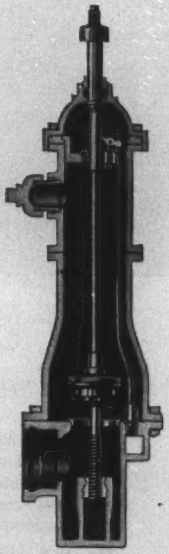

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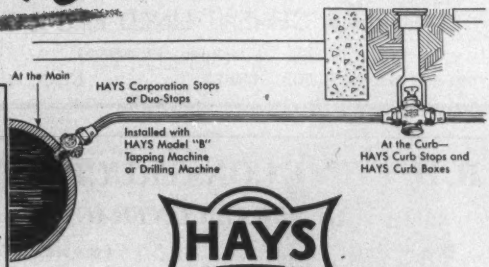

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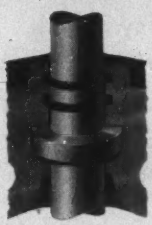

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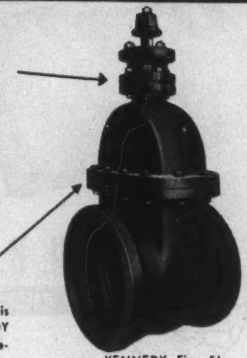

KENNEDY Fig. 56
A.W.W.A. Standard Iron-Body Double-Disc Gate Valve, with Bell Ends.

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Fig. 571

Fig. 561

Fig. 566

Fig. 566G

Fig. 566FTR

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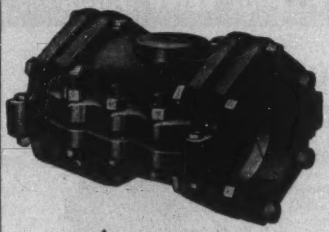

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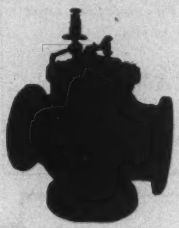

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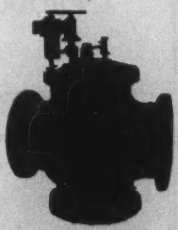

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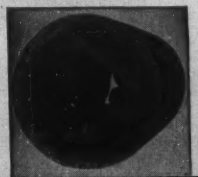

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